THE

IMPROVEMENT OF RIVERS

A TREATISE ON THE METHODS EMPLOYED FOR IMPROVING STREAMS FOR OPEN NAVIGATION, AND FOR NAVIGATION BY MEANS OF LOCKS AND DAMS

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IN TWO PARTS

PART II—IMPROVEMENT BY LOCKS AND DAMS

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IMPROVEMENT BY CANALIZATION.

CHAPTER I.

GENERAL DESIGN AND CONSTRUCTION OF LOCKS AND DAMS.

General.—By the term canalization, or slackwatering, is meant the creation of a series of pools in a river, connected by locks, and affording at all seasons of the year a depth sufficient for navigation. A few cases are found where such pools are formed by natural reefs or bars of rock, but in the vast majority of instances a dam, which may be either fixed or movable, has to be built to obtain the depth required.

There are certain conditions to be determined with all structures for slackwater navigation, regardless of whether the dams are fixed or movable. The principal ones comprise the proper selection of locations, the determination of the level of the sills and of the minimum navigable depth, and the height to be given the lift, or vertical distance from pool to pool.

Where the dam is fixed, its entire structure is brought to the full height to which it is desired to raise the water-level; when movable, the fixed part is merely a foundation on which to erect a suitable superstructure which can be raised or lowered as desired. The lock walls usually are built several feet higher than the crest of the dam in order to enable lockages to be made when the water is considerably above the normal stage of the pool.

Location.—The lift is usually the chief factor in determining the location of a lock and dam, since the new pool must not only afford the required depth on the sill of the next lock above, but must also provide navigable depth over all obstructions between the two dams unless dredging is to be employed. The actual slope of a pool is too uncertain to be taken into account in determining the location, since in seasons of low water it becomes very slight, and is always changing with

NOTE.—A map of the location of the principal systems of river improvements in the United States is given on Pl. 46.

the discharge of the river. Its flood height, however, must be considered when stationary dams are to be built because of its effect upon adjacent lands and industries, and this is then often of great importance. In movable dams the case is different, because the dam can be lowered or partly lowered upon the approach of floods, and the river can thus be restored, in whole or in part, to its natural condition before the water has risen much above the ordinary pool level. For this reason the height of the pools of movable dams can in some locations be fixed to suit special conditions, such as the placing of the works on a desirable foundation, or the avoidance of dividing the harbor of a city into two parts, etc.

In any work for creating slackwater a careful survey of all the available sites should be conducted, and the location made at the place having the greatest advantages. The principal desiderata where the lock and dam are both to be built in the open river, and adjoining each other, may be summed up as follows:

1. A satisfactory natural foundation upon which to build the works, low enough to afford the proper channel depth below the lock, but not low enough to make it difficult to reach or expensive to build upon, and stable enough to reduce to a minimum the dangers of undermining.

2. A straight channel for some distance above and below the lock, so as to provide an easy entrance and exit for boats. This can often be secured best by making the location near the middle of a long stretch of straight or nearly straight river.

3. Banks of firm material, such as clay, rock, etc., and of at least the average height of those along that part of the stream so that they will not be easily overflowed or cut away. This is particularly desirable at the abutment end of the dam.

4. Sufficient level, or nearly level, land on the lock side of the river to form a yard for purposes of construction, and which will provide sites for the locktenders' dwellings.

If the location is to be at a ripple or shoal, then the works should be built so as to make deep water on the shallows; that is, the lock should be below the shoal. In general, it is desirable to locate the works as far below the head of the pool as economy in construction will permit, so as to provide a good depth for navigation just below the lock.

Some engineers prefer a location for a lock "just under the point," that is, just below a bend on the convex side, on the theory that the upper entrance will not fill with drift. Unless the bend be slight, however, a lock so located is difficult of entrance from above and may prove dangerous, as, should a boat become disabled in rounding the point, she might drift over the dam before a rope could be got ashore. The upper entrance also fills with sediment more readily in the quiet water found below a point. Where the bend is very pronounced and the stream narrow it is especially difficult to enter or depart with a tow from a lock so situated. This type is usually known as a "point" location.

Other engineers prefer a location directly opposite, that is, on the concave side of a river, in order to reduce the risks to the entrance and exit of craft. This, however, tends to draw the drift into the upper approach, but there is usually less deposit of sediment than with a location on the convex side. This type is usually known as a "bend" location.

The location of a lock at a bend has the advantage that the river is usually wider at such a point than in a straight reach, and the dam will consequently restrict the waterway to a less degree. Moreover, there is usually deep water along the concave bank, which is favorable to maintaining a good channel. Where the bend is slight, so as to afford a slightly curved channel above and below the lock for a distance of several hundred feet, there is but little objection to locating a lock, as far as navigation is concerned, either on the concave or on the convex side; the tendency of drift and floating bodies to obstruct the upper entrance being the most undesirable feature of the former, and the deposit of sediment the most undesirable feature of the latter. In streams which do not carry an objectionable amount of sediment a location for the lock on a slightly convex bank is usually preferable to any other, as it provides a straight entrance and exit for boats with a minimum of excavation. It is under all circumstances highly desirable, and in fact necessary, to afford safe and easy navigation at a lock, especially for tows, as these are difficult to handle in restricted waters, particularly when descending in any current. Difficult entrances, even when accidents are avoided, are a source of constant trouble and annoyance. DeLagréné, in enumerating their disadvantages, gives as one the fact that they cause the boatmen to use deplorable language.

To illustrate the advantages and drawbacks of point and bend locations we will quote two examples from a river in the United States. In one case the lock had been located just below a sharp point, and towboats entering or leaving the upper entrance, except in low-water season, had to put out lines to trees on the bank in order to escape being carried out into the river and over the dam. Passenger boats usually made a flying entrance, striking broadside on a large timber crib which guarded the upper end of the approach, sometimes succeeding in entering and sometimes being swung out by the current. In the latter case they had to take their chances of successfully turning in the middle of the river, where there was a strong current toward the dam. This entrance was much troubled by the sediment deposited in the quiet water under the lee of the point.

In the other case, the lock had been located in a sharp bend, and, owing to the curvature, boats had similar trouble in entering or leaving, though there was of course much less current driving toward the dam. In certain stages of water, when drift was running, it was swept into the upper entrance and piled up till it lay in a solid mat eight or ten feet deep, and much of it had to be cut into

pieces before it could be removed. The deposit of sediment, however, was always slight.

With movable dams the pass must always be placed on the channel side, or on the natural location for navigation. For a similar reason the lock is placed adjoining the pass, and hence on the channel side also—that is, either in a slight bend or preferably in a straight piece of river.

In the earlier constructions and in some later ones, the entire river was dammed at or near the upper part of a sharp bend, and a cut-off or lateral canal was dug across the bend from the pool thus formed.* In this canal the lock was built, generally near the lower end, to obviate its being filled with sediment from below. Another arrangement, which has been frequently adopted in Europe, is to fix the location where one or more islands divide the river, placing the lock in one arm and the dam in the others. The disposition has generally been to place the dams near the head and the lock near the foot of the island, but some very good works have been arranged the other way. The selection of such a site for a lock and dam, whether at the upper or lower end, or in the middle, and which arm shall contain the lock and which the dam, must be decided in each particular case by a study of the locality and of the conditions affecting it.

In the United States the usual practice is to place both the lock and dam in the open stream adjoining each other, as lateral canals have given much trouble from deposits of sediment; but there have been occasional cases where a canal has been found best suited to the situation. At rapids or long stretches of river or steep slope, it is necessary to build canals connecting the quiet water above and below.

Investigation of Site.†—In the preliminary surveys necessary to determine a location, soundings and borings should be made at all proposed sites, and after the final one has been agreed upon, the whole area to be covered by the lock, the dam, and the abutment, should be thoroughly gone over with drills, and the depth and character of all materials determined. A dredge is sometimes used to assist in these investigations when the bed-rock is overlaid by a deposit. Test pits or wells should also be put down at suitable points on the banks. Where there is no rock, or where its depth is great, borings may be made by driving lengths of pipe, fitting the sections together as they are driven down, or in light material a water jet can be employed.

To bore into the rock in sedimentary streams, a large pipe may be driven and the sand removed therefrom with a sand-pump. When the pipe has been driven to bed-rock and pumped out, the ordinary drill or diamond drill may be used to ascertain the thickness and character of the stratum. Borings should be made into it at several places for a depth of eight or ten feet, since it frequently happens

* Such canals are known as "derivations."

† See also page 344, and after.

that a top crust covers a layer of softer material (see Fig. 144*a*, p. 380), and unless the presence of the latter be discovered in time, it may lead to annoying if not dangerous results. A sufficient number of holes should be put down to determine fully the profile and character of the foundation. When no bed-rock exists within a reasonable distance, or where the material found is unsuitable for a foundation, it will probably be necessary to build on piles, in which case the borings need not be so numerous nor so complete.

After the borings and soundings have been completed, and maps and profiles made, the exact position of the lock and dam may be determined and the plans prepared.

Acquisition of Land.—When the site has been decided upon, the necessary land must be acquired by purchase or by condemnation. This is usually one of the most tedious portions of the work, since titles have to be examined, heirs located, etc., and in many cases the only way to secure a clear title is to have the land condemned. At times delays of two or three years have elapsed before proper title deeds could be obtained and construction begun.

On the lock side, the amount purchased should be sufficient to allow plenty of yard room during construction, as well as good sites for the dwelling-houses. It should extend somewhat above the end of the upper guide wall (which is usually made about as long as the available length of the lock chamber), and as much further as may be necessary for excavating an easy approach. In most cases a distance of 300 to 600 feet will meet these requirements. Below the lower guide wall, unless the banks are of stable material and the river not subject to high floods, there will be needed a narrow strip from 800 to 1200 feet in length, as it has been found that the wash from a dam will erode an alluvial bank for long distances.

On the abutment side, it will usually be enough to purchase a width sufficient to extend 50 or 100 feet back from the top of the bank after the latter has been finished to grade. Above the abutment the property may continue from 100 to 300 feet, depending on the local conditions, and below it, the same distance as on the opposite bank.

In addition to the above, a right of way should be purchased from the lock to the nearest public road, so that communication will always be open.

In the majority of instances, too little land is secured, under the impression that it can be purchased later, if necessary. In such cases, the usual result is that the neighboring property becomes badly damaged from wash from the dam, causing perpetual friction with the owners, and that for this or other reasons the land has to be purchased after all, involving additional legal expenses and complications in examining the titles, and a higher price charged for land. It would appear to be much the wiser plan to buy a sufficient amount at first, since the extra cost, except where the lock is located in a city, is usually a very small percentage of the whole cost of the construc-

tion. In some cases an additional advantage has been found in having a larger area on the lock side of the river, where the dwellings for the employees will be located, in that the temporary buildings of the contractor for the works, generally erected on land rented from adjacent owners, may become the homes of very undesirable neighbors after the contractor has abandoned them, and it is desirable for that reason to have as much space as possible between such habitations and the houses occupied by the lock employees.

All corners should have monuments of stone or concrete suitably marked, and the land, if of any value for cultivation, should be fenced.

Arrangement of Lock and Dam.—The question next to be decided after determining the location is upon which side of the river the lock shall be built. If rock is found at the site, it is not often available for use on both sides of the stream. In such cases, other things being equal, the question arises whether to build the lock or the abutment upon the rock. In many streams it is necessary to place the lock far in the bank in order to avoid restriction of the waterway. If the rock rises out of the water and forms the bank on one side, as is frequently the case, it would be expensive to excavate it for the lock and its approaches, and in such cases the abutment is usually located on that side, especially since, both with fixed and movable dams, the bank below the abutment is the one that is most exposed to undermining from the currents. If the lock is located accordingly upon the opposite shore, its foundation must be carried down below danger of undermining; this in some cases will also entail much expense, and a careful comparison alone can show the most economical location.

With movable dams the undermining effect of the water upon the lock foundations is usually not great, because the pass, which is placed adjacent to the lock, is not opened until the head has been materially reduced by the opening of the weir; but with fixed dams there are currents in all rises which will disturb the river-bed to considerable depths.

In situations where the foundation is all good or all bad, the lock should as a rule be placed upon that side of the river which will afford the easiest ingress and egress to boats.

No fixed rules can be made to suit all cases, and each location must be worked out anew, because ideas which would be applicable to one might lead to unfortunate results in another. The general desiderata have been stated on page 335; they can rarely be all fulfilled, and the final decision has in most cases to be a balance between the favorable and the unfavorable factors.

Navigable Depth.—The minimum depth which should be given a system of slack-water improvements is regulated by the draft of the largest craft when loaded, to which must be added not less than 6 inches to a foot for clearance, with shallow-draft navigation and not less than $1\frac{1}{2}$ to 2 feet for a navigation of deep draft.* In earlier

* See foot-note, page 65.

days a depth of 4 feet was considered sufficient; later, this became too small, and 6 feet were required. This depth is now largely used both in the United States and abroad, but in its turn it has become insufficient where there is much traffic to be handled economically. Thus the Seine navigation of 6½ feet channel depth had to be increased to 10½ feet, as described further on; the Rhine in its lower portions was similarly modified; and the Ohio River canalization, commenced for a 6-foot depth, has been remodeled within the last few years so as to provide 9 feet. It is desirable, therefore, as far as can be done in designing the structures, to provide for the requirements of future commerce which may be developed by the slackwater system. A district whose commerce might originally have been satisfied by a navigable depth of a few feet may develop manufacturing, or become a producer or purchaser of coal, ore, etc., which will require for economical transportation an increased depth of water; or, if the stream in question is tributary to one having a greater depth, it may be desirable to extend this depth to the tributary in order not to break bulk in transit. It is much better to provide a depth of water too great for immediate wants than one believed to be just sufficient. Commerce will be quick to take advantage of it, and if the district possesses any natural wealth or possibilities of development the wisdom of the course will soon become apparent.

A valuable lesson in this respect may be drawn from the Seine, where the first system of slackwater between Paris and Rouen was established between 1838 and 1853, at a cost of about $2,800,000, affording a depth of water of 5¼ feet.* Soon after its completion the improvements in railroad transportation caused serious inroads on the river traffic, and it finally became evident that if the latter was to be kept in existence the system must be enlarged. Accordingly, between 1858 and 1878 new locks and dams were built, and the capacity of the old ones increased to afford a depth of 6½ feet. The cost of these changes was about $2,800,000. The relief, however, was only temporary, and between 1878 and 1888 a further expenditure of $12,200,000 was incurred, securing a depth of water of 10½ feet, and the result was the creation of an immense traffic on the river, and a general development of the valley. Even this depth was at length considered inadequate, and about 1910 a project was submitted for increasing the depth to 23½ feet, with the object of making Paris a seaport.

A similar example has been afforded by the St. Mary's Falls Canal, in Michigan, between Lakes Superior and Huron, through which passes most of the interstate commerce of the Great Lakes. The first canal with its lock was completed in 1855 at a cost of about a million dollars. In 1870 enlargements had to be commenced, which were finished in 1881, and cost over two million dollars. These soon proved insufficient, and in 1887 more extensive work was begun, including a lock 800 feet long and 100 feet wide, which was completed in 1896 and provided

* "Canalisation de la Seine," Boulé, 1889.

a depth on the sills of 21 feet. The cost of the last improvements was $3,700,000, but nevertheless it became evident that further accommodation would have to be provided before many years, and about 1910 excavation was begun for a third and still larger lock, which with the Canadian lock, will make four locks available at this point for commerce.

Similar demands are being made at other points in this country, and it is becoming apparent that if river traffic is to hold its ground many of the existing systems must be enlarged. At present most of the new locks in the United States are designed for a channel depth of 6 feet, but it would be better wherever practicable to provide for 8 or 9 feet, so as to avoid rebuilding and altering if greater channel depths are subsequently required.

Size of Lock.—It may be taken as an axiom for similar reasons, that it is better to build a lock too large for present needs, than only large enough. The trend of modern transportation is toward cheap rates, which means larger boats and greater draft of water, and if rivers are to hold their own in competition with railroads, it will be necessary to improve them with this in view. A railroad must renew its rails and rolling stock because of wear, and can then make them suitable to modern demands, but a lock once built can only be changed by a special and very heavy expense, which could have been avoided at the outset for a small part of the cost.

In one instance a lock was built which proved of practically little value, since the only boats which could utilize it had to be of small size and tonnage, and could not carry much freight, and the expenses were consequently too high in proportion to the profits. Several boats attempted to establish a trade, but all met with the same causes of failure, and as a result the traffic on the river was gradually reduced to that of floating timber. The lock in question was 27 feet wide in the chamber, and of a length just sufficient to contain one barge.

The widths of chamber used on rivers in the United States are usually 27 feet, 36 feet, and 52 or 55 feet, and on the Ohio 110 feet. These widths are based on the size of a standard coal barge, which at present is 25 to 26 feet wide, and 125 to 135 feet long. The chamber is usually made to accommodate one barge length for the smaller locks and two for the larger locks. The Ohio locks are about 600 feet in available length. A lock of the first width (27 feet) should not be built, unless under very special circumstances, since it is too small to accommodate profitable steamboat traffic. It is much better, wherever practicable or where the future development of commerce may seem to warrant it, to build to the width of 55 feet, with a good depth of water on the sills, even if existing locks on the same river are smaller. It should be remembered that the extra width merely involves some additional excavation, and some extra expense for longer miter walls and gates, whereas if the chamber is later on found to be too

small, it must either be enlarged at serious expense, or it will always be a cause of undesirable delays to navigation.

The latest locks on the lower Seine have maximum chamber widths of 55 feet 9 inches (reduced at the gates to 39 feet) and available lengths of 497 feet. The channel depth is $10\frac{1}{2}$ feet. On the upper Seine the corresponding sizes are 39 feet and 590 feet, with channel depths of $6\frac{1}{2}$ feet; on the Oise, $39\frac{1}{3}$ feet and 410 feet, with a similar channel depth.*

A few old locks have been enlarged in this country; but where new ones have been built, even on streams having small locks, larger ones have frequently been put in. In France many locks have been torn down and rebuilt with greater capacity.

The practice of building the chambers with entrances narrower than the basin has not obtained in America, although it is often to be met with in Europe. Thus on certain new locks on the Moldau the entrances are 36 feet wide and the basins $65\frac{1}{2}$ feet wide. This saves material in the gates and cross-walls, and also avoids much excavation of the approaches, but it delays the handling of traffic, as the boats, if in tows, have to be moved sideways in the chamber.

Large locks, on the other hand, while they have advantages, have the inconvenience of wasting water and consuming time in filling and emptying the chamber where only a small lockage is to be made. Generally this waste of water is of little consequence, but there are times on some rivers where the supply of water is insufficient and must be saved as far as practicable. An intermediate pair of gates is sometimes introduced, and this not only obviates the objections named above, but has also the additional advantage of allowing the continuous use of the lock during repairs to the other gates. Another method is to build a small lock adjoining the large one, of a size to accommodate single boats and the smaller tows. The locks at Bougival, finished in 1883, and at Suresnes, near Paris, are of this type. Where a fixed dam is to be constructed adjoining the lock, the smaller rivers are not usually sufficiently wide to admit a double lock, but in wide streams and where movable dams are to be built this plan has many advantages.

Lift.—In determining the lift of a lock and dam, that is, the vertical distance to the crest of the proposed dam from the crest of the next one below, due regard must be had to the riparian property and to any adjacent water-power mills or industries. The heights of bridges crossing the stream may sometimes have to be considered also. If navigation alone is to be consulted then the fewer the dams the better, within certain limits, because each is an obstruction which causes more or less delay to transportation, besides expense for maintenance, and by increasing the lifts the number of locks and dams can be reduced with advantage. However, the sizes of the parts to be maneuvered must be kept well within the limits of

* See tables at the end of the book for general dimensions of locks and dams.

the power or apparatus available, and the effect which the flow of a great volume of water will have on the works themselves and on the banks below, with the consequent danger from undermining or cutting around, must also be considered. The effects upon the régime of the river, in causing shoals or bars which may become obstructions, must not be omitted from the study. With proper care in the selection of the locations, and proper study in the design of the various parts of the works, these objections can usually be overcome.*

In France, where practically all the dams are movable, their lifts are generally small, although in the later dams it has been increased considerably. In the United States the lifts of the fixed dams are rarely less than 10 feet and in several cases as much as 18 feet, a low-water lift of 41 feet being found on the Tennessee River, near Chattanooga, and one of 63 feet, adopted in 1912, on the Black Warrior River, in Alabama, at Locks 17 and 18. This lift will be overcome by a flight of two locks. Where the banks along the stream are high, there may be no objection to using a high lift, provided proper precautions are taken in the construction, one of the chief troubles where floods are high consisting in the rough water and strong currents which will result just below the dam, and necessitate ample bank protection as well as safeguards to boats in approaching the lock. With movable dams the lifts have usually been made small, rarely being over 8 feet, although in some of the later dams, as on the Seine, Moldau, Big Sandy, and Mohawk Rivers (see Chapters VI and VIII) lifts have been employed up to 15 feet.

With rivers the lift usually has to be kept moderate because of flooding property, etc., but with canals, where such objections in most cases can be controlled, there are many examples of locks of very high lifts. Thus the lock at the head of the St. Denis Canal in Paris has a lift ranging from 32 feet to 36 feet; the canal around the rapids of the Columbia River in Oregon has two locks, each of about 34½ feet lift; several locks of the New York State Barge Canal have similar lifts, and one lock is of 40½ feet lift; on the Mississippi River at Keokuk, Iowa, a lock of 40 feet lift was completed in 1911; on the Tennessee River, as before mentioned, is a lock of 41 feet lift; and on the projected Georgian Bay Ship Canal in Canada (1910) lifts up to 50 feet were recommended. The new locks at Henrichenburg on the Dortmund-Ems Canal and Minden on the Ems-Weser Canal have lifts of 46 feet, and the lock under construction in 1912 at Georgenfelde on the Masurisch Canal in East Prussia will have a single lift of 65 feet.†

Height of Lock Walls.—The height of the walls above the crest of the dam should be such that by the time a flood has almost risen to their tops the stage

* For a general discussion on the factors determining the economic lifts and economic navigable depths for a system of slackwater see article by D. M. Andrews in vol. L, Transactions Am. Soc. C. E., 1903, entitled "The Economic Improvement of the Coosa and Alabama Rivers." For existing lifts see page 688 and after.

† The locks of the new Welland Ship Canal (Canada) have lifts of about 46½ feet.

of the river will, in theory, permit boats to pass over the dam. In France, they are usually built to or just above the level of the highest navigable water, and this rule is followed on some streams in this country where navigation ceases at a moderately high stage, but it is not applicable to our larger rivers where boats sometimes run at very high stages.

With movable dams in the United States the coping is usually placed from 3 to 5 feet above the upper pool; with fixed dams it is made from 9 to 12 feet, except where the flood range of the river is very small, when it is placed lower. With movable dams abroad, where the floods rise comparatively slowly, the distance is sometimes as small as 18 inches. The object of having an ample height of wall—technically known as the guard-wall, or guard—above the upper pool, where the dam is fixed, is to allow the river to rise to a considerable height before it can drown the lock and thus interfere with navigation. Theoretically, when this occurs, boats should be able to pass over the dam, but practically it is a condition not always realizable except at an expense quite incommensurate with the benefits to be secured, since it may require an excessive height of wall. This is especially the case with rivers from the mountains, where the rises are very rapid. However, in such streams, by the time the locks are drowned, the current has usually become too swift and dangerous to allow boats to run, and, at the most, navigation is suspended at ".drowning-out" stages for a short time only each year.

In some rare cases, lock walls have been built a few feet lower at the lower end than at the upper end, for the purpose of cheapening construction. Such a design, however, is rarely applicable in practice.

When the waterway has been restricted by the construction of the dam, it is necessary to have a higher guard than where a long spillway has been provided; otherwise, the structure will "go out of lock" much sooner than it should.

Borings.—The most satisfactory way of determining the character of the excavation and of the foundation is by digging test pits. Where these are more than a few feet in depth, however, they are apt to be expensive, since the sides usually require cribbing, just as in sinking an open well. If quicksand is encountered, moreover, it is very difficult and often dangerous to carry down the excavation. In comparatively dry soil holes bored with a 2-inch auger will also give satisfactory results for shallow depths. Certain types of post-hole augers can be used with advantage, and have been applied with success in obtaining material from a depth of 21 ft. in a water-soaked soil consisting of layers of sand, gravel, and clay. They can also be used for obtaining samples from a river bed by driving a casing and putting down the auger inside it.

The next simplest method of investigation is by means of drive-rods. These are usually of 1-inch drill steel, a few feet longer than the expected depth of the

hole, and should be made of one piece, as welded rods break easily. They are sharpened to a blunt point and driven down by sledges, trestles and planks being used for a working platform if the rod is long. In dry earth rods of different lengths can often be used, the first one being pulled up and a longer one substituted, and so on. With average material these rods can be driven to a depth of 30 or 35 feet and at a rate up to 50 lineal feet in 8 hours with 3 men. The character of the driving affords a rough idea of the material encountered, but the chief value of the method lies in its showing quickly and cheaply whether rock or hard strata exist. Jack-screws should be provided for pulling up the rods, as in some cases they bind tightly. A modification of this method is to use solid $1\frac{1}{2}$-inch rods in 5-foot lengths, joined with screw couplings and with a steel pointed shoe. These are driven down by a light drop-hammer as described for wash-drilling, and are more rapid and powerful in operation than the hand-driven rods.

In light gravel and in sandy material the presence of hard strata can usually be determined very quickly by using a steam or gasoline force-pump and a water-jet. The pipe may be of 1 inch or $1\frac{1}{4}$ inches diameter, nozzled down about 50 per cent. Hand pumps are of little use for this method, as they do not supply enough water nor give enough pressure.

Wash-drill borings have to be used for depths or conditions where the foregoing methods are not applicable. A casing is first driven down as far as possible. This usually consists of extra heavy $2\frac{1}{2}$-inch pipe, cut into lengths of not more than 5 feet, and provided with extra heavy couplings and an open shoe of hard steel. It is driven down by a drop-hammer weighing from 150 to 175 pounds, working in leads like a pile-driver and operated by 4 or 5 men pulling directly on the hammer rope. The top of the casing is provided with a cap and the hammer has a hardwood cushion on its striking face. The most convenient method of working the hammer is to use a tripod of 4×4 inch timbers about 16 feet in length, loose-bolted at the top, and with the leads hanging therefrom. When driving is suspended the leads and hammer are merely pushed to one side, and if the whole apparatus is to be moved, it can easily be lifted by the men, or be taken apart and loaded on a wagon. The drill proper consists of 1-inch gas-pipe in lengths of 12 feet or over, with a tee-head and a hose connected to a force-pump, which is usually worked by two men. The lower end of the pipe is provided with a special drill-point perforated with small holes. Through these the water is forced, and as the pipe is lifted up and down the drill-point loosens the material at the bottom of the casing and the water flows up and carries out the particles. These afford an indication of the character of soil penetrated, but it is not always reliable, as in certain kinds of clay, for example, the finer material is dissipated, and only the coarser particles appear. In such cases, a sample of the material in place should be obtained by removing the drill point, putting on

a 2-ft. length of pipe instead, and driving it down into the undisturbed stratum, so as to secure a core. The driving of the casing and the drilling are carried on alternately, the casing being kept as far ahead of the drill as practicable, so as to keep out surrounding material. Where boulders or cobbles interfere with the driving of the casing, half a stick or more of low-grade dynamite can be placed at the bottom of the hole and be exploded by a battery, after pulling up the casing 2 or 3 feet. Where water is scarce, the supply can be used over and over by catching the outflow in a barrel. The various parts described can usually be obtained from any manufacturer of well apparatus, and a supply of spare shoes, pipe and other parts should be always kept on hand, as in difficult material occasional breakages are unavoidable.

This method of drilling will penetrate almost any material except rock, and has been used in borings for locks, etc., up to depths of more than 65 feet. In loose material, however, the water frequently escapes at the bottom of the casing, (even where the drill is below water level) and as the loose particles are not washed up further progress is stayed. About the only remedy is a more copious supply of water, and if this fails, another location must be tried.

Where drilling in water, two boats or pontoons may be lashed together, the drill and casing passing down between them. These pontoons should be not less than 20 feet long by 6 feet wide by 2 feet in depth, as the casing is sometimes difficult to pull up, even with powerful jack-screws, and a small pontoon would be forced under the water. If they are put together with bolts, they can be taken apart and transported overland; this possibility is at times of much convenience. Spuds should be provided at each end of the pontoons for use in shallow water, and in deep water anchors must be used to hold them steady. The borings for the Ohio River locks and dams were made by calyx and diamond drills operated by gasoline or steam engines and carried on barges built for that especial purpose.

If indications of rock are met with, it is of prime importance that their reliability should be ascertained beyond all question, especially if the structure is intended to rest upon the rock surface. Drilling in gravel, as well as in material containing cobbles or boulders, often gives apparent rock indications, especially when some depth has been reached. In one case a layer of hard clay was reported as rock, and the error was not exposed until the layer had been uncovered. Cases have also occurred where apparently solid rock proved to be only a crust of rock a foot or two thick, with a layer of clay or sand underneath, and the actual bed-rock was many feet below.* In some of these examples, contracts had been made on the superficial indications, and the actual facts were only discovered when it was too late, necessitating changes of plan and much additional expense.

* See also "Foundations-Rock," p. 378 and after, and Fig. 144a, p. 380.

For these reasons the character of the rock should be ascertained beyond all doubt, as well as its average surface elevations under all the proposed structure. If the rock is seamy, as is often the case with limestone, information as to the thickness, etc., of the seams can be obtained by taking a rod of ½-inch iron, pointing it, and bending the point at right angles. By scraping the point along the sides of the drill hole it will catch in the seams and their thickness can then be measured.

It must not be supposed that because rock is found at or near the ground level on one side of the river, it necessarily continues at or near similar elevations to the other side. In many cases the rock slopes away sharply, or where exposed in one bank it frequently disappears abruptly out under the river bed. It has often been found also that where rock ledges entered a high bank and might be expected to rise as they went further from the river, the opposite was the case and the rock dipped away sharply.

Sufficient borings should be put into the rock for a depth of 10 feet or more below the expected foundation to remove all doubts as to its character. In sandy ground a pipe may be driven to the rock surface and the material inside be removed by a sand-pump or a water-jet; in harder material the wash-drill and its casing may be employed.* Drilling can then be done by ordinary methods, jointed pipe being used for the drill handle where the rock is deep. If trouble arises from sand running into the pipe where it meets the rock, it can sometimes be checked by ramming down stiff clay, or by pouring in cement, agitating it at the bottom, and letting it set until it forms a plug which the drill will penetrate later. This method of ascertaining the soundness of the rock, while rough, has usually proved quite satisfactory for all ordinary investigations for locks and dams. Cases are rare where the expensive core-drills are necessary, although their use in this class of work is increasing. Where a great head of water has to be dealt with, as with reservoirs, they are often needed for exhaustive information, but for the comparatively small heads of navigation works, equally serviceable and reliable information as to the presence of seams, etc., can usually be obtained by hand-drilling under the direction of an intelligent foreman. The diamond drill is the oldest type of core-drill, but is somewhat expensive, as a single bit or cutting shoe costs between $1500 and $3000, and requires a skilled setter to keep it in order. A later type, known as a calyx drill, uses hard steel shot as the cutting medium under a revolving tube and has given good satisfaction. Drills of this class were used in the borings for the Ohio River system of locks and dams. Part of them were equipped for boring and part for ordinary drilling. The drill parties

* A simple form of sand-pump can be made by taking a 2-ft. length of pipe of diameter suited to the casing. A ½-inch iron rod a few feet in length, with its lower end swelled into a bulb large enough to close the bottom of the pipe and its upper end attached to a cord, is placed in the pipe and the apparatus is then lowered to the bottom of the casing. By jerking it up and down the loose material is worked into the pipe, which is then hauled up and emptied.

consisted of three men each, working on a boat with a steam or gasoline engine for power. During the season of 1909 the cost per foot in sand and gravel averaged $1.03 and in rock $2.84.

Every care should be taken to verify the indications of the borings, and frequent personal examination of them should be made during their progress. Mistaken deductions are usually expensive and may at times even lead to dangerous results, and the engineer should give every attention necessary to secure reliable data.

Cost of Borings.—The cost per foot run of a total of about 75,000 lineal feet of borings taken between 1905 and 1908 for the New York State Barge Canal was as follows: *

Core drilling in rock, 84 cents to $30; average, $2.54; do. in earth, gravel, etc., 23 cents to $5.50; average $1.10.

Wash drilling in rock, 26 cents to $1.19, average $1.00. In earth, gravel, etc., 8 cents to $2.55; average, 25 to 50 cents.

Tests pits in earth, about 31 cents.

Auger holes in earth, 5 cents to 33 cents; average 9 cents.

Drive rod holes, 4 cents to 10 cents.

The wide variation in some of the above, as for instance, in the cost of core drilling in rock, was frequently due to the presence of oblique seams or of boulders, which turned aside the cutting edge and made it very difficult to get any hold on the rock. With the wash drilling a frequent cause of expense was the great depth of some of the holes, running up to 60 feet and over. Estimates for drilling should always be liberal, as it is impossible to make close figures on the probable cost.

Cofferdams.—The temporary structures built to keep the water from inundating the site during construction are known as cofferdams, and may vary from a simple ridge of earth to a costly structure of cribwork, according to the local needs. A reliable cofferdam is very necessary for river work, on account of the shortness of the working season, and the delay and expense of work is interrupted by too copious leakage or by weakness in the structure. Thus on the Ohio River the low-water season is sometimes less than two months in duration, and in order that this may be used to full advantage it is necessary to build the cofferdams strong enough to keep out ordinary floods and to withstand the drift and currents of the high floods of winter and spring during several seasons. The cofferdam is thus ready for use whenever the favorable moment for work arrives, and if of suitable design will last several years with slight repairs.

Owing to the temporary need of these structures, engineers and contractors are often tempted to use too much economy in their construction, to their subsequent regret. A broken cofferdam is usually tedious to repair, and if the damage occurs during the working season, it disorganizes the work at the very time when

* See also Engineering News, January 17, 1907.

progress is most to be desired. Cases have not been rare in which the contractors lost from a third to a half of the working season because of flimsy cofferdams, and were in consequence put to a much greater expense than would have been entailed by a suitable construction at the outset. It is the opinion of many engineers that where a lock or dam is to be built by contract, plans and specifications for the cofferdams should be included in the agreement, and that their original cost and the cost of all repairs should be paid for directly as a part of the contract. Where the work is liable to be let to the lowest bidder—and such bidders are usually inexperienced, especially on river construction—it is believed that this plan offers many advantages. The accumulated experience of engineers with cofferdams on rivers in the United States is extensive, and they should be able to select and plan the type most suitable to the work in question. All bidders would thus be placed on an equal basis, and the high percentage usually added to bids because of risk with cofferdams, as well as the certainty of loss to the contractor if he does not know the risk he incurs, are both eliminated. Where this method has been followed it has in general proved more satisfactory to all concerned than the other system which the contractor designs and takes all risk with the cofferdams. Under the latter plan he usually loses one or more cofferdams before he is willing to build one sufficiently strong; a needless waste of labor and material is involved; and often the completion of the work is postponed for a year or more, to the loss of all concerned.

In some recent contracts, as on the New York Water Supply, the risk of pumping has also been eliminated, the contractor being paid by the million gallons for all water pumped, the price being adjusted where necessary according to the lift. This plan has been found to work very satisfactorily, as the contractor receives a fair return for his work, but receives no pay for work not done, as would be the case if he bid for much pumping and found later that very little was required.

General Principles of Design and Construction.—Having selected one or other of the types described further on, the dimensions and location of the cofferdam has next to be decided. This is determined chiefly by the slopes of excavation inside and the amount of contraction of the flow. If the cofferdam is too near the proposed construction, there is danger of its being undermined from within as the excavation proceeds. Ample room should therefore be allowed, as it is expensive and inconvenient to have to rely on bracing to keep the cofferdam from falling over. Where the excavation is not deep and is not in sand or running material, it is usually sufficient to assume a slope of excavation inside of 2 horizontal to 1 vertical, with 10 feet or so for leeway. Where the excavation is in soft material, or space is limited, sheathing can be provided to hold up the material, and the construction should then be carried on in short sections, and the footings

be put in as fast as possible so as to reduce the time the excavation has to be kept open.

The contraction of the river should be kept as small as practicable, since a scouring of the bed outside takes place in proportion to the area blocked up (except of course with rock), and if construction has to be carried out later upon this outside portion, the natural bed may be found to have scoured many feet below its original level. At one lock and dam on the Ohio River, where the contractor had built cofferdams reducing the area of discharge about 50 per cent, the velocity in the open portion of the river was increased at a 10-foot stage from 3 feet to nearly 10 feet per second, and the bed scoured to rock, 20 feet below low water. In other examples where the contraction has been made nearly as great as this, the slope of discharge between upstream and downstream arms has reached as high as 3 feet. In addition to this danger of scour, the river may cut into the opposite bank seriously if it is left unprotected. A case occurred within recent years where part of the bed and of one bank were eroded from these causes to an extent which made necessary considerable and expensive changes in the plans and in the contract. Except in unusual cases the total contraction of area when the river has reached the top of the cofferdam should not exceed about 30 per cent, and should be as much less as practicable. Protection of the bed against scour can be secured by using sand-bags or riprap, but this is usually expensive, and it may be cheaper to let the scour proceed and backfill later if necessary with material similar to or better than that washed out.

The top of the cofferdam is generally placed from 5 to 10 feet above low water, but where the working season is very short, as on the Ohio, the top is often raised to 14 or 16 feet above, so that work will be stopped only by unusual floods. This additional height of course affects the contraction referred to in the preceding paragraph. Sluices should always be provided so that the enclosure can be flooded before the river overtops it. If the water is allowed to pour over the top into an empty cofferdam (the engineer or contractor usually keeps the enclosure empty until the last moment, when there is not time for leakage to fill it before the river overtops it) it will quickly undermine part of the structure, and in a short time a breach will occur which may take a long time to repair. These sluices may be made 4 or 5 feet in depth below the desired level for flooding, and 20 feet or more in total width, according to the area of the enclosure. They can be closed by loose 1×12 inch planks set vertically, and wooden flumes should be provided to carry the water down into the enclosure where there will be no danger of a washout. The material around the sluices and under the flumes should be protected where necessary by gravel and riprap against wash from leaks or from the strong currents which occur when sluicing.

The usual plan of a cofferdam is as shown in Fig. 129, running directly out

into the river, and with right-angled corners. These cause side-currents as shown by the arrows and result in pronounced scour at B and C, and a swift current along BC. The parts near B and C should therefore be protected by riprap; for this purpose one-man stone, if angular, will usually suffice. The side BC may also need similar protection. It is a good plan in exposed cases to build rough cribs filled with stone at B and C, as shown in Fig. 130; these will prevent drift and ice from battering down these portions of the cofferdam, and will throw the scour-holes shown in Fig. 129 further away from it. The cribs can be set on the river-bed with plenty of riprap thrown along their river faces; they frequently settle, but rarely enough to cause trouble. The arms AB and CD need no protection

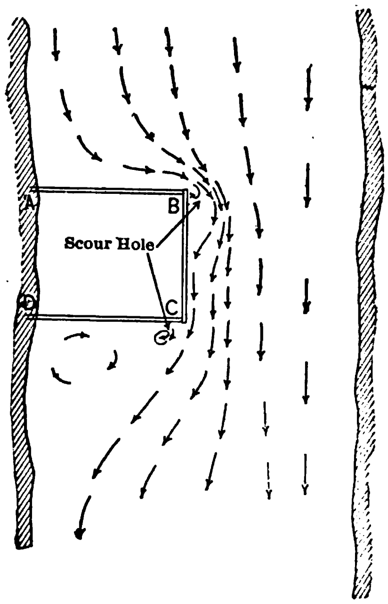


FIG. 129.—Directions of Currents around a Cofferdam.

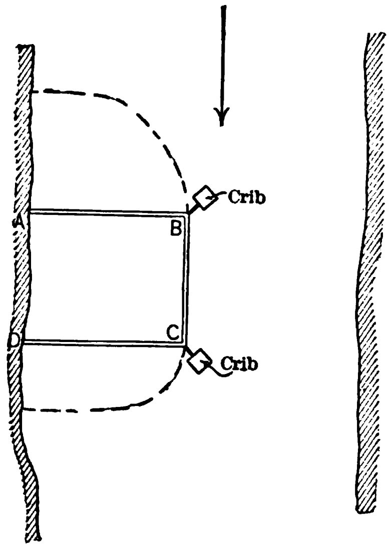


FIG. 130.

except at the corners as just described, unless conditions are unusual. The side-currents and scour at B and C can be reduced somewhat by curving AB and CD as shown by broken lines in Fig. 130, but this would entail a longer and more expensive cofferdam. The bank ends of the arms AB and CD should be sloped up to 2 feet or more above the general level of the top of the coffer, and should be well protected with one-man riprap, especially if on a concave bank, as high water always tends to cut around them.

The material used for filling or backing a cofferdam admits of wide range. The best is composed of gravel and earth, as it packs tightly and if a leak occurs the gravel will eventually stop the washing away of the earth. Gravel and sand dredged from the site have often been used successfully, but if the sand is coarse the leakage may be copious. Clay, if not too hard or lumpy to pack closely, is

excellent, and in some cases fine sand has had to be used, but it is difficult to keep it from running out, and a cofferdam filled with it is liable to lose much of the material, especially when first pumped out, and to leak badly and become weakened in consequence. Suitable material can usually be obtained from the banks or the bed close at hand, and by using it generously, especially on the outer faces, the results should be satisfactory. After a rise the cofferdam will usually be found to be considerably more watertight than when new, as the silt fills up the small spaces and the water settles the earth together. If the leakage at first is too copious, the usual remedy is to use more backing.

The material used for backing (see Figs. 132 to 134) is usually similar to that used for filling, and should be protected with riprap where exposed to strong currents. In some cases where a cofferdam has been of unusual width, or where it has been on rock in confined situations and exposed to strong currents, the backing has been omitted and without harm resulting. In most cases, however, especially with the box type of cofferdam described further on, in which the sheathing is usually driven but slightly into the bed, it is necessary to use backing so as to reduce the tendency of the water to force its way under the structure. In one example of a box cofferdam, about 8 feet wide and subjected to a head of about 6 feet, backing was omitted from a portion of the work, and this part undermined soon after unwatering was begun, while the remainder, which was provided with backing, stood unharmed. The river bed was of very sandy gravel.

In building the cofferdam it is usually best to complete first the river arm *BC* (Fig. 129). This will cause very little contraction and will enable the work to be carried on without much increase of current. After *BC* has been finished and the corner *B* protected, *AB* or *CD* can be begun. The building of either one will at once affect the current, and if construction is carried on from one end only, scour will begin in the opening between *A* and *B* or *C* and *D*, and will increase as the arm is built out until great difficulty may be incurred in the final closing. To avoid this, the part of the river-bed in which scour is taking place can be protected with sand-bags or stone, or where practicable a temporary dam of earth or sand-bags can be thrown up on the upstream side, encircling the scour-hole and cutting off the current. This work has to be carried out quickly, and the temporary dam must be begun all along and be kept about level as it is built up, or a washout will occur. The best method, where it can be done, is to begin all the cofferdam between *A* and *B* or *C* and *D* at the same time and build it up evenly, so that no concentration of the water will occur. With sheet-pile cofferdams part of the earth backing can often be placed first so as to cut off the flow, the piling being driven through it later.

With large cofferdams, however, it is often very difficult to control the scour. The problem was solved in an ingenious manner with the cofferdam for the dam

across the Charles River, at Boston, Massachusetts. The river was subject to a tidal variation of several feet, with strong ebb-and-flow currents across the site. Sheet-piling was first driven all across the bed and protected with riprap, so as to form a sill. Above this lines of piles and bracing were put in, and a series of wooden drop gates fitted between. When all was ready, the ropes suspending the gates were cut, and the gates dropped on to the sill, closing the river in a few seconds. Earth backing was immediately thrown in, and the cofferdam completed. The discharge of the river itself, being very small, was cared for by the usual methods.* (See also p.358, third paragraph.)

During construction the possibility of a flood and consequent loss of part of the unfinished cofferdam should always be borne in mind, and the building should therefore be carried on so as to expedite specially those portions which would be most liable to damage while uncompleted. The most vulnerable points are at B and C, along the river face of BC, and also at those points between A and B, and C and D where the arms AB and CD have not been completed.

It is sometimes desirable to leave open part of the downstream arm of a cofferdam so that a dredge can enter and commence the excavation. The gap can be closed by extending the arms or by putting in a bank of good earth.

Where a cofferdam depends upon its weight for stability, as with the crib or the box type, a rule sometimes used is to make the width from 75 to 100 per cent of the height. This rule appears in general to have worked satisfactorily in practice, although the horizontal shear is sometimes excessive, when considered under the condition that the lower portion of the filling is usually immersed, as the cofferdam always retains a high saturation line. With a slippery filling, as clay, there is frequently a gradual inward leaning movement of the structure from the base up, due presumably to the low resistance to shear.

Pumps.—(See also "General Notes on Construction," p. 381 and after.) For cofferdams of ordinary size, say up to about an acre in area, two 12-inch centrifugal pumps will usually be found sufficient, and after a short "seasoning" of the cofferdam only one of these will be needed. The second pump should always be provided, however, for use when the first one breaks down, or when the head is increased by floods. A 6-inch pump is also very useful for special points. For cofferdams of very small size, as for an abutment, a steam siphon will generally serve, and this can sometimes be dispensed with if the head is very small, and one or more men with buckets or hand-pumps be employed instead. One of the pumps should be set in a protected place far below the top of the cofferdam, so that pumping can be started as soon as the flood recedes to a suitable elevation. The limit of lift by suction should be about 20 feet or less. In large cofferdams, especially in gravel, additional pumps will be needed. Thus, at the Assiout Dam

* Engineering News, November, 1908.

on the Nile, built on a silt foundation honeycombed with springs, nine centrifugal pumps from 6 to 12 inches in size were employed at one time in a sand-bag cofferdam enclosing from 2 to 3 acres, and even this number at times proved insufficient.* On the Ohio River at Dam No. 18, set in gravel, the pass cofferdam of an area of about 2 acres required three 12-inch centrifugal pumps, and the weir cofferdam of about 3 acres required four similar pumps, when the river was 12 feet above low water. The extreme lift was from 33 to 38 feet. In the cofferdam for Lock No. 1 on the Allegheny River, set on porous gravel, six large pumps were needed.

With cofferdams on a porous material, such as gravel, one feature of the pumping is often encountered which deserves notice. In beginning the unwatering the pumps usually have no difficulty in lowering the water several feet. Then they fail to gain on it, and for several days and sometimes longer the water stays at about the same level. Thereafter the pumps gain on it again and usually can complete and maintain the unwatering without further difficulty. The reason for this phenomenon appears to be that below a certain level the ground-water stored in the neighboring land is drawn upon by the pumps as from a reservoir, and until it has been sufficiently exhausted it will continue to supply more than the pumps can remove. This theory has been tested and verified by pipe-wells driven for the purpose. In some cases a slight increase in the pumping capacity will assist matters, as in one case on the Mohawk River with a gravel foundation. The contractor by using one 10-inch pump had drawn the water down to within 2 or 3 feet of the final level, but could lower it no further, although he pumped for nearly a month. After cleaning out the boiler, however, the pump lowered the water in a few hours and there was no further trouble thereafter.

Ample pumping capacity is essential to the progress of the work, and it is far better to have an excess than a deficiency.

Earth Cofferdams.—The simplest type of cofferdam consists of an earth bank, and where conditions are suitable for its use it is usually the best. In cases where the structure is set into the bank, as often with abutments, the excavation can frequently be thrown out in a ridge and be made to serve without any further work (Fig. 131). Comparatively little leakage will come through it, and the use of sheet-piling is superfluous except in unusual cases. It must be remembered that ordinary sheet-piling is not water-tight, unless it has been driven with great care and in soft material; the water will find its way through the joints and often in considerable quantities. The real coffering material in ordinary cofferdams is the earth used to fill it and back it. The sheet-piling, however, in places infested with musk-rats, is of much value in preventing these animals from burrowing through the material.

* Proceedings Inst. C. E., vol. clviii.

This type is very useful where repairs are to be made to a fixed dam, as it obviates the necessity of drawing off the pool and stopping navigation. It has been used where an entire dam had to be torn out of the bed-rock and rebuilt, one section at a time, during a season of many and unexpected rises, requiring the crest of the cofferdam to be raised several times to prevent the river from overflowing it. Where used under such conditions the corners (corresponding to *B* and *C* in Fig. 129) should be protected with plenty of riprap, as should also the outsides of the arms running parallel with the flow, or they may be cut away by the concentrated current.

Sand-bag Cofferdams.—These consist of sacks or bags filled with earth. Any strong material will serve for the sacks if it is not coarse enough to let the filling seep out, and in the United States gunny-sacks are usually employed. They should be not more than two-thirds full, or they will not pack together. The

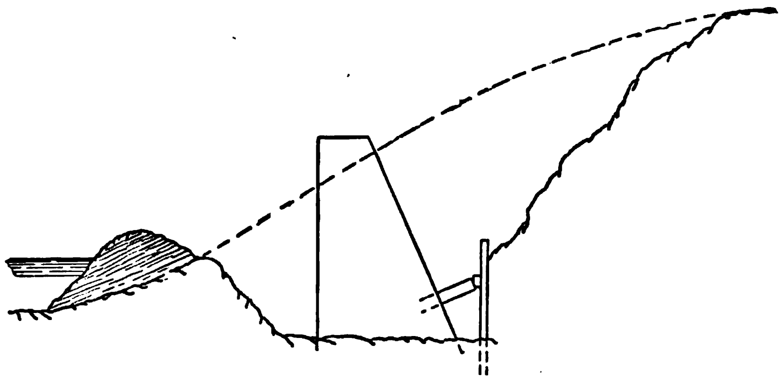


FIG. 131.—Cofferdam of Excavated Material.

filling should be preferably of loam, although a mixture of sand and gravel has often been used. At the Assiout Dam on the Nile this type of cofferdam (called a "sudd") was used entirely, the bags having to be filled with river silt. This was so fine that the strong winds sucked it out of the exposed bags in clouds of dust, and some of the slopes flattened to an inclination of 6 horizontal to 1 vertical. Parts of these cofferdams were more than 30 feet in height, and held a head of 20 feet of water, while one of them enclosed an area of about 7 acres. A total of 2,697,000 sacks and 642,370 cubic yards of material was used in the cofferdams.*

This type of cofferdam is especially useful in strong currents and shallow water, as the faces can be built up vertically. For a very small head a width of the length of one sack is sufficient if the joints are packed with earth; for greater heads greater widths must of course be used. In the former case the bags are

* Proceedings Inst. C. E., vol. clviii.

usually placed as stretchers; in the latter case they are placed in rows of stretchers and headers alternately, earth being packed into the interstices of each layer where practicable as the building is carried up. The life of the bags is usually limited to a year or two, as they soon begin to rot. Their cost may vary from about 8 to 15 cents apiece, according to circumstances.

Steel Sheet-pile Cofferdams.—Steel sheet-piling can in certain cases be used advantageously for cofferdams, as it requires only a small space, and if braced from the inside will hold a considerable head of water. The outside earth backing can sometimes be dispensed with, and where leaks occur through the joints above the ground level boiler ashes or fine gravel can be sifted into the water and the particles will temporarily close the spaces. The piling, however, is expensive, and may cost from 50 cents to a dollar or more per square foot in place, but if carefully handled it can be withdrawn and be used again. This feature in special cases may result in ultimate economy.

FIG. 132.—Pile Cofferdam, Single Row. FIG. 133.—Pile Cofferdam, Double Row.

Pile Cofferdams.—These consist of a row of round piles with sheet-piling and walings or of two rows of similar construction. (Figs. 132 and 133.) The piles are usually placed about 10 feet apart longitudinally, and 10 to 16 feet crosswise, according to the head of water. The walings are usually of 6×10 inch to 10×12 inch timbers, and the sheet-piles are formed of two or three planks each 2 inches thick and 10 or 12 inches wide. A single thickness of matched plank is also used where suitable. The round piles may be driven 6 to 8 feet into the ground and the sheet piles 4 feet or over. Where two rows of round piles are used each bent is tied together with a 1- or $1\frac{1}{4}$-inch iron rod, the walings and sheet-piling being placed on the inside so as to hold in the earth filling. In more recent practice two or more lines of 6×8-inch walings have been used, one above the other, with $\frac{3}{4}$-inch tie rods, and a single line of 2-inch plank battened with 1-inch strips has been substituted for the sheet-piling. To reduce leakage it is generally necessary to back the outside with earth, and riprap has to be used where needed for protection. Where the flood currents are strong the space between the two rows has sometimes to be planked over on the top or has to be riprapped to prevent the filling being washed out during rises. The single-row type has proved about as satisfactory for ordinary cases as the double-row type, and is a good deal

cheaper. The chief advantage of the latter is that it has more mass to resist the effect of drift and strong currents, and that the filling is less liable to be washed away. As stated above, the earth filling and backing is the real coffering material, and hence the chief function of the planking is to hold in the filling. For this reason it has been found that the single thickness of battened plank just described, when set a little way into the ground, has given as good results, both in the single- and double-row pile cofferdam, as the more expensive sheet-piling.

These pile cofferdams require a sufficient depth of natural bed in which to drive the piling, and where this exists they have been successfully used for heads of water of 15 to 18 feet, even in light material, and they withstand rises excellently. They require plenty of space, however, since they must be set well away from the edge of the excavation in order to prevent loss of support during the removal of the inside supporting earth. The type costs from $10 to $16 or more per foot run of cofferdam, according to circumstances.

FIG. 134.—Crib Cofferdam Built of Logs.

Crib Cofferdams.—These consist of rows of round or sawed timbers, built up to form cribs or pens (Fig. 134). The pens are usually 10 to 12 feet long, and 10 feet or more in width, according to the head of water. The inner faces of the longitudinal rows are covered with 1-inch plank, the joints of which are battened with strips to retain the filling. The plank should be spiked very lightly, for ease in subsequent removal. Additional planking is sometimes placed outside on the river face, but in most cases it is not needed. Where sawed timber is used, the pieces are usually 8×8 inches or 10×10 inches, drift-bolted together. If the coffer is of round timbers, these consist of saplings of a maximum diameter of about 6 inches at the small end. They are also drift-bolted. There is no need to butt the longitudinal pieces at a panel point; they can meet anywhere provided they break joints, and there is no need of splicing them. The round-timber cofferdam is usually the cheaper and is equally serviceable as the one of sawed timber, and has in addition the advantage of allowing the filling to pack more closely around the pieces. With sawed pieces the filling does not pack well against the under sides of the ties, and troublesome leaks often occur along them in consequence unless the cofferdam is well backed with earth on the outside. Where this earth backing cannot be used, it is best to use round timbers, tamping the filling around and under them as

far as can be done. The cribs are usually built up in the water, the weight gradually sinking them to the bottom. Where the cofferdam has to be placed in swift water, the lowest portions of the cribs are usually begun in quiet water and are then towed into position and held by lines until they have been built up and filled sufficiently to stay in place. The sections towed should be small if the current is strong, as they are hard to handle in swift water. The cribs are set directly on the natural bed; occasionally a trench is dredged for them, but this is rarely necessary. Where the ground is uneven, the bottom of the crib is built to fit as described in the chapter on "Fixed Dams."

It has been customary to drift-bolt together the various pieces, necessitating the injury or destruction of much of the timber when the coffer is removed, on account of its having to be torn apart. With round timbers this is rarely of consequence, as after two or three years' service the saplings become partly rotten, but with sawed timbers, containing much heart wood, the pieces are still in excellent condition and could be used for other purposes. To avoid this waste, a few crib cofferdams have been built in recent years where the pieces were fastened together by long 1-inch bolts. These bolts may run from bottom to top, or from the bottom to just above the water line, where another series of long bolts begins and continues to the top. They are run through the timbers, and act in the same way as drift-bolts. This method has proved very successful, as it costs little more in construction than drift-bolting, and the timbers can be saved uninjured when the cofferdam has to be removed.

Where the cofferdam rests on rock, and has no backing outside, leakage and washing out of the filling often occur along the bottom. In quiet water a strip of canvas placed along the outside and kept in place by sandbags will usually stop this. Where this cannot be done, the planking may be sharpened to a long wedge point and be driven down against the rock with mauls. The blows force the thin wood into the irregularities of the surface, and usually secure a good fit.

The final closing of the arms *AB* or *CD* (Fig. 129, p. 351), where built out from the ends, is often difficult because of the concentrated current. On a cofferdam on the Hudson River on a rock bottom, this part of the work was accomplished by building and partly filling the closing crib (20 feet in length) on 12×12-inch timbers over the opening. A hole was then bored near each end of these timbers and a stick of dynamite exploded simultaneously in each one, causing the crib to drop into place. Another way is to spike vertical guide pieces on the ends of the cribs already in position, and to build the closing crib in the gap where it will be held in the current by these pieces. The crib is suspended by ropes which are slacked off as the timbers are built up, thus lowering it gradually into place between the guides. (See also p. 352.)

The cost of a crib cofferdam runs from $15 to $25 or more per foot run, depending on circumstances.

Box Cofferdams.—These are a modification of the pile cofferdam, and in many cases they can be used with a good deal of economy. They consist of horizontal

waling-pieces, which are supported by temporary uprights spaced according to the pressures. Inside are placed vertically planks of $1\frac{1}{2}$ to 2 inches in thickness (Fig. 135). Tie-rods pass through the wales to prevent spreading, and the box thus made is filled with gravel, clay, etc. The planks are driven slightly into the river-bed, and the outside should have plenty of backing to prevent the water forcing its way underneath. Where the cofferdam has to be placed in a considerable depth of water the uprights, walings, and tie-rods, are usually put together on a barge in sections perhaps 20 feet in length. The joints of the walings are scarfed and placed in the same vertical plane, the tie-rods passing through them. This makes a hinged joint between each section and allows the sections to be pushed off the barge and be righted in the water, providing at the same time continuous lines of walings. Longitudinal and cross-bracing is used to keep the sections from warping, and they are weighed as needed to keep them in place. The remaining planking is then put in, and the filling is completed. If the filling

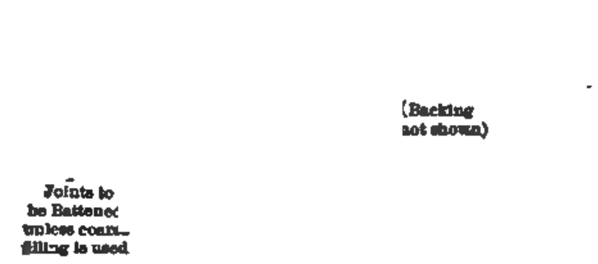


FIG. 135.—Box Cofferdam.

is of plastic material, as clay, it is usually necessary to use some cross-bracing even when the cofferdam can be built directly in place, as the placing and the settling of the filling are very apt to force the structure out of plumb, thus reducing its stability. In one case where a soft clay filling and no cross-bracing was used the cofferdam leaned to an extent which rendered necessary very heavy struts in order to prevent it from collapsing.

A cofferdam of this type was built for Lock 48 on the Ohio River near Evansville, Ind., in 1912, enclosing about 20 acres. The largest sections were 20 feet wide and 26 feet in height above the river bed, and designed to sustain a head of water of 24 feet. Ten lines of walings were used on each side, ranging from 2×12 inches to 10×10 inches, and the tie-rods ranged from $\frac{3}{4}$ to $1\frac{1}{4}$ inch in diameter. The sheathing consisted of one thickness of 2-inch plank. As the river-bed was of sand, a line of triple-lap sheet-piling was driven along the river face and backed with sand-bags, gravel and rock, to guard against scour. The cofferdam was filled with material taken from the river-bed, but one 18-inch centrifugal pump was sufficient to keep down the water when the river-level was about 18 feet

above the bottom of the enclosure. Parts of the structure had to be built in water from 25 to 28 feet deep.

This type of cofferdam has been largely used on the Ohio River locks and dams and has been found both economical and satisfactory. After the boxes have been built and filled they are floored over with plank or riprapped so that overtopping floods will not disturb the filling. In a few cases the covering has been made of concrete instead of timber. The boxes near the shore and in shallow water are usually 10 feet wide, while those for deeper sections are 16 to 20 feet or more in width. The cost per linear foot ranges from $10 for the narrow box to $20 or more for the wider boxes.

Choice of Type. (See also preceding paragraphs).—Damage to a cofferdam usually results from undermining on the river side, or from breaches due to water pouring over the top and scouring out the filling or washing away the supporting earth, or from impact of drift, ice, etc. Failures from direct pressure or from the water forcing its way under the cofferdam have not often occurred. In selecting a type, therefore, the liability to floods and the probable duration of the work must be chiefly considered. The location must also be taken into account, as a cofferdam on a convex bank is less exposed to injury from currents, drift, etc., than one placed on the bend or concave side, where the water is always swifter, and drift and ice more liable to run.

Where the water is shallow and the work can be carried out quickly, the earth or sandbag cofferdam, or a combination of the two, will often serve. These kinds are cheap and easily repaired.

In deeper water the pile coffer, of single or double rows, can be used. The double-row type is less easily damaged, as its mass is self-supporting, and for rivers subject to frequent and high floods it is preferable to the single-row type, unless the latter can be provided with plenty of backing as a protection.

For high cofferdams with limited room inside, or where the cofferdam is to be set on rock, or where the river is of a dangerous character and the work may last through several seasons, the crib cofferdam or the crib combined with the pile type have usually been employed. This is the most expensive type, but it is also the strongest, and where well designed and built it has proved able to endure for several seasons the worst conditions with very few repairs. Within recent years, however, the box cofferdam has been used very successfully in similar situations, as on the Ohio River, and has the advantage of being more speedy and economical in construction.

General Calculations for Structures.—The following paragraphs refer to the general conditions of loading to be dealt with in designing locks and dams; the special conditions, as the stresses in the various elements of a lock, of a movable dam, etc., will be found described in the corresponding chapters. Typical sections of the chamber walls of locks will be found on Pl. 45*a*.

Underpressure.—No unusual complications are met with in the theoretical analyses of the strains in or produced by the walls; the three main factors are the weight of the

wall, the pressure of the earth backing where such is used, and the effect of the water. The types to be dealt with in ordinary practice consist of abutments, masonry dams, guide walls, and lock walls with their various subdivisions. An element of uncertainty exists in the calculations, however, due to the fact that with natural foundations of material porous to any degree, the amount of underpressure from leakage or seepage from the head of water is unknown. With a foundation of dense rock, such as a granite without seams, it appears probable that under moderate heads there will be no under pressure unless it can penetrate along the joint between the masonry and the rock, and many walls have been built on the softer rocks without allowance for underpressure (as may be seen from some of the examples on Pl. 45*a*) and are still standing satisfactorily. Modern practice, however, is tending more and more towards security against possible weakness, and recent experiments with high heads appear to show that even a sound rock will in time transmit the water pressure.* Experiments made by the U. S. Coast and Geodetic Survey with hollow balls of thick glass submerged in the deep sea showed that the water was gradually forced into them under the extreme pressure. The joint between the masonry and the rock, and the joints in the masonry itself, may also be carelessly made. Any leakage into these joints or seams will produce an upward pressure, and if the mortar is porous, the flow in and out as the water rises and falls and the varying pressure will tend to increase the effect. Examples are known where such conditions have come to exist in old lock walls of stone which were not designed for such underpressure, and where signs of stress have become apparent in the course of time.†

With a porous foundation, however, the case is different. Thus, with a lock built on gravel the pressure of the upper pool forces the water beneath the foundation and behind the land wall under more or less pressure until it escapes as best it can into the lower pool. There appears to be little information from experiments on actual structures as to how much the head is reduced during the passage, nor as to how much it may vary between different parts of the walls, and it is a subject about which, like earth pressures, only general assumptions can be made. (See Chap. IV, Fig. 197.) The underpressure with a gravel foundation is no doubt quite different as regards extent and uniformity from that with a foundation of dense clay, and local variations in the material itself doubtless have more or less effect on the results. It is certain, however, as shown by leaks which have appeared from under walls, and by others which have occasionally come from the banks just below structures, that the water passes under and behind the walls, wherever it can force a passage. The only safe method seems to be to take assumptions which will make the walls safe under the worst probable condi-

* For opinions regarding underpressure on masonry dams see the Proceedings Am. Soc. C. E., January, 1912, and the following issues; for experiments on high-lift dams, see "Engineering News," July 31, 1913, page 202.

† See also "Foundations-Rock," p. 378 and after.

tions, or which have been approved by actual practice, and to make the design accordingly. The few experiments on record seem to show that the total underpressure is considerably less than such assumptions would give, and in fact there are several canal locks in the United States, built within recent years on pile foundations driven in sandy material, where the element of underpressure was disregarded entirely because of financial reasons, and the bases of the walls made about 45 per cent of their height. So far the walls appear to be standing satisfactorily, but the matter in general is one of too much uncertainty to permit of undue chances being taken. DeMas states that, in general, for lock walls backed with earth and with a coping width of 4 feet and sloping backs an *average* thickness of wall equal to 40 per cent of the height has been found to give ample stability. This would, in most cases, give a larger base width than 45 per cent.*

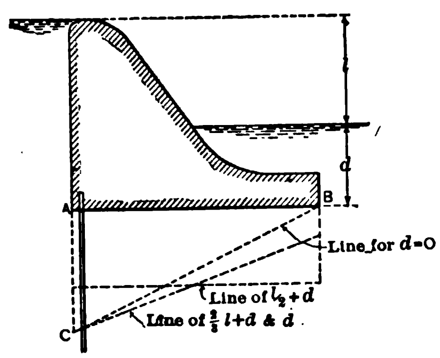


FIG. 136.

The following limits are given as representing the general practice in the United States. They are based on the opinions of various designers, and while some engineers may consider them too conservative or may question the distribution of the pressure, and others may consider them not conservative enough, it is believed that they represent the general consensus of practice as existing to-day, and that they are within safe limits. The total effect of the various forces acting will be discussed later under paragraphs referring to the different structures.

Let l be the lift between pools and d the depth of the lower pool above the foundation. For a masonry dam on gravel, clay, etc. (Fig. 136), assume an underpressure at A due to a head $l+d$, diminishing to d at the down-stream edge or toe at B, or use $\frac{2}{3}l+d$, distributed uniformly over the base. For rock use $\frac{2}{3}l+d$ and $\frac{l}{2}+d$, respectively. (See also page 365.) Many designers assume the pressure at A as $\frac{l}{2}+d$ for rock instead of $\frac{2}{3}l+d$, with d at B. The values assuming uniform distribution do not appear, however, to coincide with actual conditions as far as determined by experiments, and are therefore of doubtful applicability. Under extreme conditions, that is, with the lower pool drawn off, the underpressure at B is usually assumed as zero, as shown by the line CB, thus shifting the center of pressure nearer to A, the pressure at A being taken as before. The foregoing assumes a line of sheet piling or a cut-off along the upstream edge or back of the dam, and a free escape for seepage at the toe. The downward pressure of the lower pool on the top of the apron of course affects the net total of underpressures for the normal conditions.

* "Rivières Canalisées," p. 285.

As the pools rise in a flood, l and d change, and their effect on the stability is sometimes more unfavorable than with the water at normal levels. This point should always be investigated.

For an abutment, assume that $ABCE$, or the part equal to the width of base of the dam, is subject to underpressure from a head equal to $\frac{2}{3}l+d$ for gravel, etc., and $\frac{l}{2}+d$ for rock (Fig. 137) for both normal and extreme conditions, that is, with both pools full, and also with the lower pool drawn off, respectively. The assumption is made the same for both cases on account of ground water usually being present in the bank. For $ECDH$ assume that the underpressure comes from a head equal to d only, unless some unusual conditions exist. For earth pressure on the back of $ABDH$, assume that the line of saturation reaches from the base to FG, a line joining the upper and lower pools. The foregoing supposes that a line of sheet-piling or a cut-off wall runs under the upstream face of the wing wall into

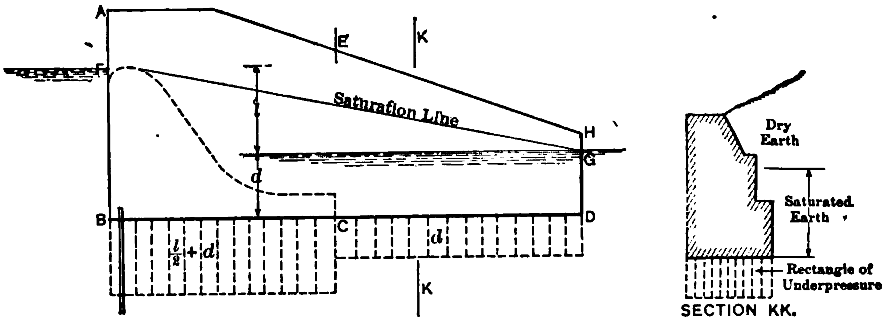


FIG. 137.

the bank. The underpressure can be taken as distributed uniformly across the base for the sake of simplicity, as shown in Section KK. As with the dam, the effect of varying pool heights should always be investigated.

For lock walls, if sheet-piling or a cut-off wall is used around the base, and the dam is at the upper end of the lock (Fig. 138), assume an underpressure on the portion $ABCD$ above the toe of the dam equal to $\frac{2}{3}l+d$ for gravel, etc., and $\frac{l}{2}+d$ for rock, uniformly distributed transversely, l representing the lift between pools and d the average distance from the lower pool to the base of that part of the wall under consideration. If the sheet-piling of the dam, in addition to the sheet-piling of the lock, is continued directly across the lock so as to enclose the upper end like a box, $l+d$ should be used for that portion. Similarly, if the dam is at the middle or at the lower end of the lock use $l+d$ (Fig. 139). For the river wall under the two latter conditions many designers use $\frac{2}{3}l+d$ instead of $l+d$.

For the portion $DCFE$ below the toe of the dam use $\frac{l}{4}+d$, also uniformly

distributed. Some engineers use this amount for the part between the dam and the lower gates, and for the remainder d only is used.

The line of saturation behind the land wall AE (Fig. 139a) can be taken as running from upper pool level at A to lower pool level at E, and can be

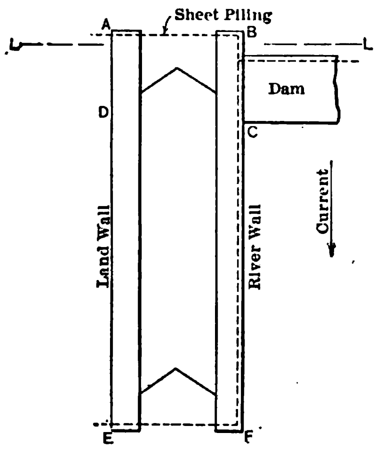
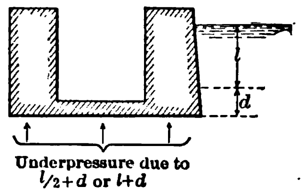


FIG. 138.



Section LL of FIGS. 138 and 139.

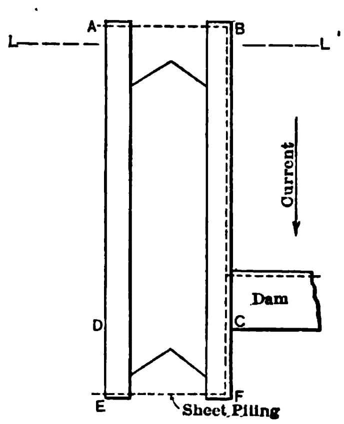


FIG. 139.

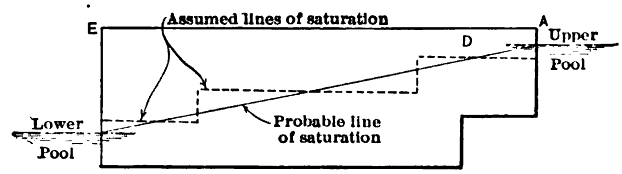


FIG. 139a.

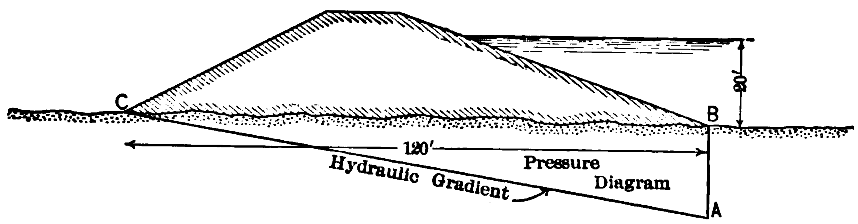


FIG. 139b.

averaged into three horizontal portions, viz., the chamber wall and the two thrust walls (Fig. 139) for the sake of simplicity. This line is usually taken irrespective of whether the dam is at the head or at the foot of the lock, unless the chamber with the latter case is a very long one.

The pressure from the upper pool under the river wall below the dam is assumed by some designers as a triangle (Fig. 139c), instead of as a rectangle, on the theory that the water pressure decreases as it approaches its point of escape along the river side.

With certain types of structures, such as the foundations of movable dams where the mass is comparatively light, the analysis of the underpressure should be more exact than the formulas outlined on p. 362. This is usually obtained by plotting the hydraulic gradient or line of underpressure, the length of the perimeter of the base of the structure being first approximated from the relation of the head to be sustained and the material of the natural foundation, as explained in the following paragraphs.

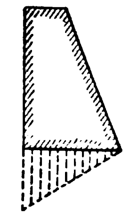


FIG. 139c.

The safety of a dam on a porous foundation, such as sand or gravel, depends on the protection against undermining from above and erosion from below. In the United States the latter cause appears to have occasioned much more trouble than the former. The head of water tends to force a passage under the dam, and the distance from the point where the water can begin to pass under to the point where it can escape freely into the lower pool establishes the hydraulic gradient. Where the distance is short in comparison with the head, and the material is of a fine nature, as sand, the flow finds an easy passage and tends to wash out the particles at the point of escape. This erosion gradually works back under the foundation until a passage is opened to the upper pool, and the dam is undermined. The further the water has to travel under a dam the less will be the tendency to undermine, and the determining factors thus appear to be the distance, the material, and the head. The following coefficients, deduced largely from observations of existing structures in India and elsewhere, may be used for these relations. They are based on the approximate equation $l=cH$, where l is the distance in feet between the points of entry and escape of the water passing under the dam, c is the coefficient of percolation, and H is the head or the difference in feet between the water above and below the dam.*

	Coefficient, c.
Class A Fine silt and sand as in the River Nile	18
Class B. Fine micaceous sand as in the Colorado River and the Himalayan rivers	15
Class C. Ordinary coarse sand	12
Class D. Gravel and sand	9
Class E Bowlders, gravel, and sand	4 to 6

Thus with an earth dam on gravel of the dimensions shown in Fig. 139b, H being 20 feet, and c being equal to 6, the base length BC would be not less than 6×20 feet or 120 feet. Where the material is fine silt, as in Class A, BC would be 18×20 feet, or 360 feet. In the case in question, B being the point of entry of the water and C being the point of escape, the hydraulic gradient would be

* Engineering News, December 29, 1910; W. G. Bligh. An examination of some of the more recent structures in the United States gave about the same values for c as stated above. For experiments on rate of percolation, etc., see Mr. Allen Hazen's investigations, Report of Massachusetts State Board of Health, 1892. A discussion on dams on sand foundations will be found in the Transactions Am. Soc. C. E., September, 1911.

represented by the line AC and the uppressure on the base of the dam by the triangle ABC, AB being made to scale 20 feet.

To secure the long distance required for the safety of structures on silt or sand, three general methods are in use, the first employing one or more cut-off walls or lines of sheet-piling, such as A (Fig. 140), around which the seepage must force its way; the next consisting of a lengthening of the downstream apron, such as B; and the last employing a clay puddle apron several feet in thickness on the upstream side, such as C. This last method combined with the first one

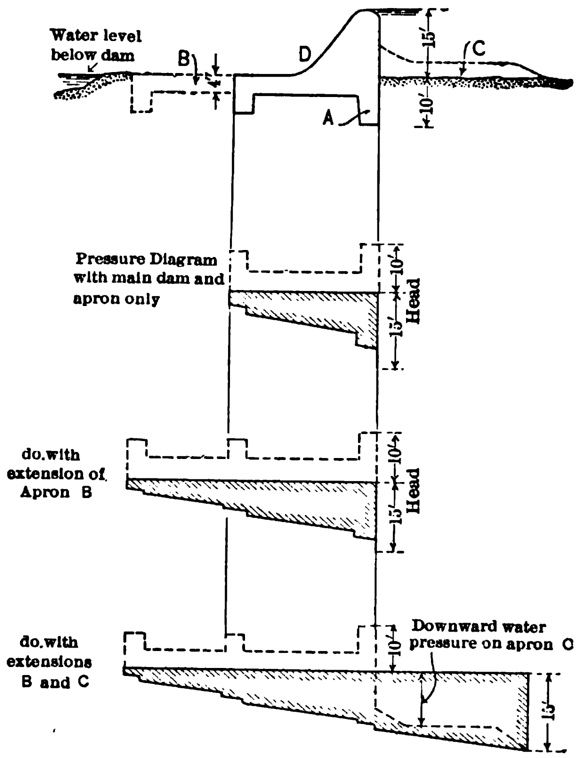


FIG. 140.—The hatched portion below the horizontal line represents the up-pressure on the base of the dam due to the head from the upper pool; the portion enclosed in broken lines represents the up-pressure due to immersion. The total up-pressure is their algebraic sum. The diagrams neglect the weight of the masonry and of the aprons.

has been used very successfully on the large dams on the Nile (see Chapter VIII), and in India. The thickness of the apron is rarely less than 4 feet, special care being taken to make a water-tight joint where it abuts the main dam. The second method has the objection that the point of escape for the water is moved further downstream, thus increasing the up-pressure under part of the base of the dam, as will be seen on reference to the pressure diagrams in Fig. 140. It is usually a good deal more expensive on that account than the method of employing an apron upstream.

Another method which has recently been successfully used on some of the Indian weirs is to use cut-off walls resting on the river-bed instead of projecting into it (Fig. 198, Chapter IV), thus dividing the fall into a series of short pools, each of which is separated from the next by the percolation distance of the base of the adjoining cut-off wall.

Where impervious sheet-piling or cut-off walls are used the water appears to work its path down one side and up the other instead of along the shortest line of escape through the undisturbed material, and the effect on the percolation length, as far as has been ascertained by the experiments of Colonel Clibborn and other engineers in India, as well as by occasional tests elsewhere, appears to be directly proportional to the length and vertical depth. Thus a single masonry or puddle cut-off wall 10 feet deep below general base and 4 feet wide will apparently cause the seepage to travel a distance of 24 feet. If two lines of sheet-piling or two cut-off walls are used, they should be spaced not nearer than twice their depth in order to secure their full effect.

The pressure thus appears to decrease about uniformly in proportion to the distance the water has to travel along the perimeter of the base of the dam between the points of entry and escape (Fig. 140). For instance, with a head of 15 feet and a total travel of 75 feet, the pressure will decrease about uniformly 1 foot in 5 feet, stepping down at each cut-off in proportion to the length of travel around it, thus according with the numerous experiments on filtration through an ordinary mass of earth. It should be noted, however, that these conditions are probably never realized in actual practice, since sheet-piling is rarely tight, and there is usually more or less variation in the density of the natural foundation. However, the theory appears to be sufficiently close to the facts to form a safe working basis in designing the structures. As water under pressure will follow a straight surface much more easily than a broken one, the under side of the masonry should be designed so that the water will have to travel around several angles in forcing its way. (See Pl. 72.)

The up-pressure considered in the paragraphs just preceding is that due only to the effect of the upper pool head trying to force its way under the dam, and there is an additional up-pressure due to the immersion of the foundation below the lower pool. Thus in Fig. 140, when there is no head and the lower pool is flush with the top of the apron, the net up-pressure on the base of the main apron will be that to due a 4-ft. head less the weight of the overlying masonry; on the base of the upstream cut-off wall it will be that due to a 10-foot head less the corresponding weight, etc. If now the upper pool is filled, the up-pressure due to seepage from it will also come into play, and the total up-pressure will be that due to the combined effects of the upper and lower pools as shown by the diagrams of the figure, less the weight of the masonry, etc. The net weight

of the masonry apron is usually made not less than $\frac{1}{4}$ or $\frac{1}{3}$ greater than the total up-pressure of the water beneath it. It should be made safe for not only normal working conditions, but also for the condition, if such can possibly occur, of a full pool above and no water below. Where piles are used, they are sometimes assumed to take part of the up-thrust due to extreme conditions. Sections of foundations from American and foreign practice are given in the cuts accompanying Chapters IV to IX.

Limits for Resultants.—A lock wall is practically a retaining wall in that it has to resist lateral pressures, but while ordinary retaining walls have only steady loads to carry, lock walls have to bear side thrusts which may vary constantly and often rapidly as the chamber is filled or emptied, and, as experience has shown, they are in consequence much more liable to settlement if there are any weak places in the foundations. Moreover, a wall which may stand satisfactorily for some years on a compressible foundation will in some cases gradually settle or lean in the course of time under the repeated changes of pressure. For this reason the resultants of the forces under normal conditions of operation, that is, when boats are using the lock, should preferably pass within the middle third of the base for a soft foundation, and not much outside of that limit for rock. Under extreme conditions, as, for example, when the lock is pumped out for repairs and some of the pressures are consequently temporarily increased, the resultants may pass at a distance from the edge not greater than one-quarter of the width of the base for a soft foundation, nor greater than one-fifth for rock. By a soft foundation is meant any foundation, such as gravel, clay, etc., which might be liable to compression under heavy loading. It is usually necessary to investigate also the maximum pressures at the edges of the base, as in special cases the one-third to one-fifth rule just mentioned would give too high a toe-pressure for bases of customary proportion. For convenience of reference the formulas usually employed for this purpose are given below.

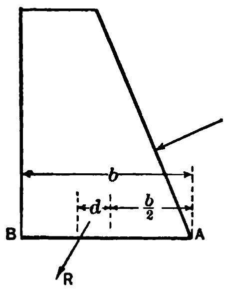


FIG. 141.

Let b represent the width of base AB in feet (Fig. 141), w the total load on AB in pounds, and d the distance from the middle of the base to where the resultant R passes. Then if R falls within the middle third, the maximum toe-pressure at B and the minimum at A are, respectively, in pounds per square foot,

$$\frac{w}{b}\left(1+\frac{6d}{b}\right) \quad \text{and} \quad \frac{w}{b}\left(1-\frac{6d}{b}\right).$$

If R falls at the one-third point, the minimum pressure at A becomes zero, and the maximum pressure at B becomes $\frac{2w}{b}$.

If R falls outside the middle third, and the structure is specially built so that A can take tension, the maximum and minimum pressures are the same as for the first case. If A cannot take tension, the pressure towards B is increased, and the maximum toe-pressure at B becomes

$$\frac{w}{b}\times\frac{2}{3\left(\frac{1}{2}-\frac{d}{b}\right)}.$$

This amounts practically to considering the wall as having a shorter base.

It is not in general desirable to allow tension to exist at A, unless it can be counteracted by anchor bolts secured to the natural foundation, since if there is any horizontal joint or crevice there into which the water can penetrate, not only will the tensile strength disappear, but an up-thrust will also result which will affect the stability of the wall.

The method of judging the stability of a wall by a numerical factor of safety, as is often done, is not very satisfactory, as it gives no indication of the toe pressure nor of the position of the resultant, and it is on these in reality that the safety of the wall depends.

Limiting Pressures.—The limiting toe pressures on the natural foundation which have been used for computing many river and canal locks built in the United States during recent years, have been as follows, the "tons" named being tons of 2000 pounds: On hard rock, 15 to 30 tons per square foot, (or about 200 to about 400 pounds per square inch), the lower amount being used for important walls subject to varying pressures; on soft rock or hard-pan, 8 to 15 tons; on gravel or good sand, 4 tons; on clay, 2 to 3 tons, or more if the clay is very hard and thick; on loam or soft earth, 1 to 1½ tons. The toe pressures in the masonry of the wall are of course the same as those on the natural foundation. Where piles have been used, they have usually been assumed to carry all the load, the bearing of the concrete on the soil being left as an additional safeguard. This load is usually limited to 20 tons per pile with good material like gravel, although higher loadings have had to be used in cases where the room was limited. In soft soils, however, it may be necessary to reduce this to 15 tons or even considerably less, while in hard material a loading of 30 tons can be carried safely.* The base of the wall must also be secured (during construction as well as after) against the tendency to slide due to the water or earth pressures. Only in rare cases, however, will this require any modification of the design; the weight of the wall and the friction at its base are usually sufficient safeguards except in soft clay, marl, or similar materials which are liable to easy displacement when wet. The following coefficients of friction may be used against sliding:

* See also the paragraphs, "Foundations," p. 372 and after.

masonry on wet clay, 0.20; on dry earth, 0.33; on rock or on masonry, 0.67. These are the values at which sliding is liable to begin, and the working values should be made $\frac{1}{4}$ to $\frac{1}{6}$ of them. Where any danger of sliding is to be apprehended—and the partially completed conditions during construction sometimes open the way to this long enough to cause it—the bottom of the wall should be stepped so as to give a hold on the soil, or other means should be used. Walls on piles in uncertain material are particularly liable to cause trouble in this respect.

Earth Pressures.—The effect of earth pressures can only be approximated, and the various formulas in vogue often give different results with the same assumptions. Owing to the varying nature of the material, which may be sandy at one part of the wall and clayey at another, the actual pressures are uncertain, and the most accurate theory will fail to determine the real loads. For this reason many

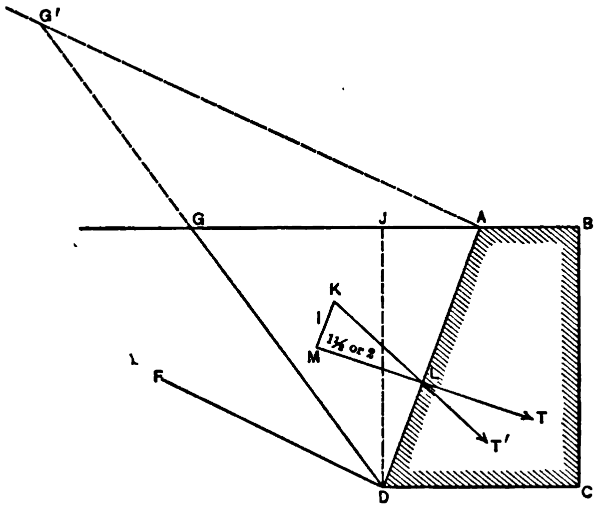


FIG. 142.

engineers employ certain empirical but simple formulas which appear to have stood the test of actual use with satisfaction, and are as follows: *

For earth level with the coping (Fig. 142) and no account taken of its friction on the back of the wall, the pressure T acts normal to AD and is equal to

$$\frac{\text{weight of triangle } AGD \times GJ}{DJ}.$$

DJ is a vertical line through D, and DG bisects the angle JDF, DF being the slope of repose of the backing. This slope is usually taken as $1\frac{1}{2}$ horizontal to 1 vertical for dry earth, and as 2 to 1 for wet earth.

If it is desired to include the effect of friction, as is usually done, the pressure acts as T' at the same point and is equal to the weight of the triangle $AGD \times 0.643$

* Trautwine's Pocket Book.

for dry earth, and to the weight of the triangle $AGD \times 0.692$ for wet earth. These equations give an apparently greater stability than the formula which neglects friction.

The angle JDA should not much exceed $26\frac{1}{2}°$, for the formulas to be generally valid.

The angle KLM of the inclination of T' to the back of the wall is equal to the angle of repose of the backing and is to be taken as $1\frac{1}{2}$ to 1 or as 2 to 1 according as the earth is dry or wet, as just described, LM being made normal to AD. If the back of the wall is vertical the triangle AGD becomes JGD.

If the backing is all of the same weight, T and T' act at L, one-third of AD above the base; if part is dry and part saturated, the weights of the separate parts can be combined, and the point of application of the resultant found. This will be below L, but is usually taken as being at L, for the sake of simplicity, its angle of inclination KLM being taken as that for earth all saturated.

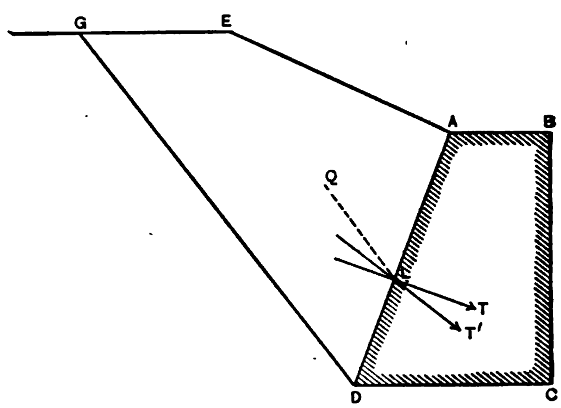


FIG. 143.

If the wall is completely surcharged the triangle AGD of the formula becomes $AG'D$ (Fig. 142), and the resultant acts similarly and at the same point as for backing level with the top, as just described. If the wall is partially surcharged, (Fig. 143), AGD becomes $AEGD$, and the point of application occurs at L where a line QL drawn through the center of gravity of $AEGD$ and made parallel to GD meets AD. This comes above the one-third point. The directions of T and T' are as before.

Rankine's Formula is also used by many engineers. It is $E = \frac{wh^2}{2} \times \left(\frac{1 - \sin \phi}{1 + \sin \phi}\right)$, where E is the horizontal earth thrust, w the weight of a cubic foot of earth, h the height of the portion of wall under pressure and ϕ the angle of repose of the backing, usually taken as $1\frac{1}{2}$ horizontal to 1 vertical for dry earth and as 2 to 1 for saturated earth (Fig. 144). The portion of the backing resting on AD is

taken as the weight of the triangle *ADG*, acting vertically. This formula is only applicable, in the form given, to walls with the backing level with the coping.

The weight of dry earth may be taken as 90 pounds and that of wet earth as 115 pounds per cubic foot. The dividing lines which may be assumed between the wet and the dry portions of the earth pressures has been described in the preceding paragraphs on "Underpressure," p. 363 and after.

Ice Pressures.—In designing works for creating slackwater, it is not customary to take any account of possible ice pressures, and experience has approved this practice. With lock walls the mass is ample to resist even severe ice gorges, and with dams there is usually a flow over the crest, even in low water, sufficient to keep the field ice above from touching the structure. The rounding or sloping of the upstream side of the crest also assists in preventing any direct thrust. (See also "Ice and Fixed Dams," at the end of Chapter IV.)

Foundations.—(See also "Foundations," Chapter V.) The natural foundation is of course a factor of the first importance in determining the general design of the lock and dam, and the limit to be set for the lift. On a rock foundation,

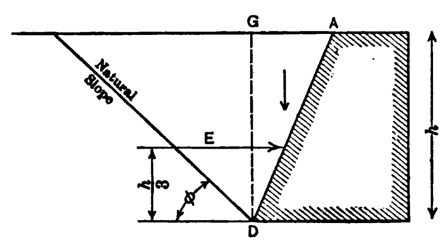


FIG. 144.

with other conditions favorable, a lift of considerable height can be used, while if the foundation be light or porous, it may be a serious risk to employ other than low lifts, especially with movable dams.

The foundation is the most important part of the entire work, as if it is improperly designed or constructed it can rarely be remedied without great expense, and any defects will invariably become manifest, though sometimes not for many years. The action of water is so penetrating and insidious that it will attack the slightest weakness, and unless the engineer can detect and secure during construction the vulnerable points of the foundation and of the banks around the structure, trouble is sure to result.

Where piles are used it has been a general practice to cap them with 12×12-inch to 12×16-inch timbers, to which a heavy timber floor is spiked, on which the masonry is commenced. It is usually much better, however, to excavate the soil for a foot or two below the pile-heads, and commence a bed of concrete on it, several feet in thickness, inclosing the piles. When this is done the weight of the wall is partly carried by the surface of the natural foundation, instead

of by the piles alone, and greater stability is thus secured against settlement. In the former style of foundation, the entire load is transmitted to the piles through the caps, and it has been found sometimes on investigation that the pressure per square inch between the surfaces of contact was greater than was safe, especially as wood immersed in water and always under pressure appears to become gradually soft. It is probably owing to the crushing of weak timbers from this cause that walls on pile foundations have sometimes cracked and settled.

Where the arrangement can be avoided, lock or dam masonry should not be built on a foundation one part of which is composed of piles or grillage and another part of the natural material alone, because unequal settlement may take place and crack the walls, if not at first, then later on when the timber has had time to show its defects. One instance of this kind was met with in a lock wall which did not show any settlement for some years, although after the first break no additional settlement became apparent.

Foundations may be divided into the general classes of sand, clay, gravel, hardpan and rock. Foundations of a pure clay are not often met with in river work, as the sand and clay are usually in combination, and may vary from a sand which contains just enough clay to make it flow like mud when wet to a clay which contains a very small percentage of sand. Similarly sand and gravel and sometimes gravel and clay are met with in all varieties of combination.

Specifications and plans should always be drawn so that the necessary variations in height, width, etc., of foundations can be made in order to meet the changes which are often found necessary when the excavation has been carried down to the proper depth.

Sand.—(See also p. 365 and after.) With foundations of sand, as well as with those of clay and sand mixed, there are two principal difficulties which must be overcome, the first, the danger of seepage or undermining from above, and the second, the danger of erosion on the downstream side. To combat the danger of seepage it is customary to use sheet-piling on the upstream side of the structure as well as along those portions where there is any risk of ground water from the bank or erosion from the river disturbing the material under the structure.* Thus, in a lock foundation where the sand is of light or porous quality, sheet-piling is usually employed on all sides except along the bank. Additional rows of sheet-piling are often placed under the miter sills, running from the river face back into the bank. In some cases (especially in European practice) the downstream side of a dam is also protected with sheet-piling so as to confine the material and prevent seepage from creating cavities under the structure. This practice, however, is not generally favored by American engineers, as it tends to produce, by confining the seepage, an increase of upward pressure under the

* See p. 377 for remarks on steel sheet-piling.

masonry. Where no such sheet-piling has been used and where the natural foundation has been of fine sand or silt, a bed of mixed sand and gravel, not less than 3 feet in thickness, has been spread in some cases along the downstream face and covered with heavy riprap to prevent its being washed out. This bed acts as a filter in that it permits the escape of seepage while holding back the fine particles of sand and silt which otherwise would be carried out by the water. In certain of the European dams layers of fascines held in place by riprap have been used for a similar purpose and appear to have been satisfactory (see Fig. 211, Chapter VI). The bridge dams of the lower Nile (see Chapter VIII), which were built on extremely fine silt, were provided with an unusually wide foundation and with a row of cast-iron sheet-piles along both upstream and downstream faces. The latter was protected from bank to bank with a layer of gravel and sand about 3 feet thick and 33 feet in width, covered with about 18 inches of pebbles, and with a bed of broken stone over the top surface. Where the width of a foundation is comparatively narrow, a considerable amount of heavy riprap is necessary in order to guard against undermining and this should be carefully watched for some time after construction and be replaced at once when washed away, as there usually occurs more or less displacement and erosion during the first few floods.

On account of the dangers of seepage and erosion with sand foundations it is customary in America to use round piles to support the masonry, since if they are employed a considerable amount of material may leak out from under the masonry without causing failure. If the sand is of coarse and firm quality it usually possesses considerable bearing power, but if of fine quality and with a certain percentage of clay, the bearing power is not always to be relied upon, as described in the next clause. It happens not infrequently, when driving piles, that a layer of gravel or other compact material is found at some distance below the surface, and if this layer is found to be only a few feet in thickness it is often best to stop the piles at its surface and not drive through it. By this method the piles will usually carry a considerably greater load than if the layer is broken through.

Sandy foundations are in general the most difficult to make secure, on account of the dangers outlined above. Thus on a movable dam on the river Oise, completed about 1903 on a sand foundation and without sheet-piling, a leak appeared under the dam soon after its construction and when the head was about 6 feet. The leak increased so rapidly that the dam had to be lowered immediately to prevent the undermining of the foundation. The resulting cavity under the masonry was subsequently filled up by drilling holes and pouring in grout, and gravel and broken stone were used to fill up the holes in the river bed. These remedies appeared to work satisfactorily. Again, on one of the tributaries of the Mississippi, where a fixed dam had been built with a movable crest several feet in height,

certain portions of the dam became undermined during floods and prior to completion, and although on piles and provided with sheet-piling one section of the dam and two of the piers were undermined and destroyed.

Where there is a movable crest a few feet in height on the dam, the minimum length W for the downstream apron required to carry the reaction of the overflow to a safe distance below the dam, measured from the toe of the dam proper, as at D, Fig. 140, page 366, may be based on the formula, deduced from observation of existing structures: $W = 4c\sqrt{\frac{H_1}{13}}$, where H_1 is the height from the top of the apron to the top of the movable crest, and c is the percolation factor, given on page 365.* The masonry can be extended downstream where necessary to secure the foundation against seepage, or an upstream apron can be used as shown by C, Fig. 140. Below the downstream apron a bed of riprap is generally used to protect the structure against undermining from below; the pieces fall over as the erosion progresses, but still afford protection for the toe. The dimensions of this bed as used in practice are shown on the various figures and plates of this and the following chapters.

Clay.—Clay is not very common in foundations for structures in rivers. When met with it may vary from the cheese-like compacted silt of abandoned river channels to the hard clay which will stand a strong current almost unaffected. This last variety is rare in river work, but it is excellent for foundations, as it is water-tight and usually of high bearing power, and in most cases the use of piles and sheet-piles can be dispensed with (in point of fact they can rarely be driven in this material) provided riprap or similar protection is used where there might be danger from excessive scour. With the softer variety of clay it is not safe to trust much to the bearing power of the soil unless it has been shown by tests to be reliable in this respect. Even when confined by sheet-piling (as should always be done on those sides of the structure where there is any possibility of the material spreading under a concentrated load) such clay is liable to flow gradually and produce a displacement of the masonry under the varying pressure of the water thrust, and during construction the weight of the banks will often force up the material in the excavation for the floor, as the weight becomes unbalanced by the operations. Long piles should be used to carry the loads and should be driven down if necessary by placing one on another until a satisfactory bearing power is obtained.

An example of settlement occurred with a lock built on piles of ordinary length driven in the alluvial soil of the Mississippi Valley. The removal of the material caused a settlement of the banks, their weight constantly forcing up

* Engineering News, December 29, 1910; W. G. Bligh. The formula appears to correspond closely with examples of apron lengths of existing dams in the United States, most of the dams, however, being without movable tops.

and disturbing the excavated surfaces, and after the side walls and floor had been completed the walls continued to settle, the tops moving backwards irregularly until they were several inches further apart at the coping than when built, and the floor also became somewhat displaced. This was of concrete 6 feet thick, flat and resting on piles, and the walls were of massive section and 51 feet in maximum height. Some years after the masonry was finished the settlement was reported to show no signs of ceasing, and the tops were then fastened together with tie-rods until the backfilling had been put in. After the completion of the backfilling the structure appeared to become stable.

A pile driven in soft clay will generally "set up" or obtain a good deal of support from skin friction after a lapse of several days, so that conditions usually become safer after the driving is finished. In one such case on the Mohawk River (Dam No. 5) the material was so plastic that some of the piles sank from 10 to 20 feet into it with the weight of the hammer alone, but after a week's interval it required several blows of the same hammer before any movement occurred. Piles driven in soft material will frequently rise just after driving, sometimes a foot or two, and the driving of adjacent piles will often force them to rise further. For this reason single-length piles are preferable to spliced ones, as even where the latter are strapped together at the joints the straps appear liable to become loosened under hard driving and the upper portion of the pile may then draw loose from the lower. Banding the piles at the joints is often necessary to prevent the ends from splitting. The loads should be kept moderate, since with high loads a slow settlement may commence after a lapse of time, such as occurred with some of the foundations of the early buildings in Chicago, resting on piles driven in swampy ground.

One danger with soft material is that any horizontal pressure on the wall will tend to make it slide forward, as the clay has little resisting power against a surface thrust; this danger may be minimized by using a continuous row of sheet-piling, or by carrying the footings down so that they will have a considerable body of the natural material to resist the thrust, or by tying the wall securely to the bank behind. In the deepening of the Erie Canal, near Syracuse, shortly prior to 1900, a portion of the cut had to be carried through a bed of marl, and it was intended to protect the sides with masonry walls supported on piles. As the work progressed, however, it was found that the material in the bottom was forced up by seepage and pressure from the banks about as fast as it was excavated, and that the walls were slowly being pushed forward. The walls were finally held in place by using long timber struts and bracing across the bottom of the prism, tied down by piles, so that the pressure from one bank was balanced by the pressure from the other.*

* Proceedings Am. Soc. C. E., December, 1899, February and April, 1900; Wm. B. Landreth.

In certain classes of saturated sandy clay the bottom of the excavation becomes soft and quaking. It can often be made stable again near the surface by digging pits or drainage ditches close together so that the water can seep into them. A layer of dry gravel and sand has also been used with success, the water apparently being drawn up into the gravel and making the material firm again. Small riprap worked into the material has also been used satisfactorily.

Gravel.—Next to hard clay and rock a thick bed of gravel is one of the most satisfactory foundations for slackwater structures. Although there is more or less seepage when under a head of water the material acts as a filter and the leaks come out clear, showing that there is very little danger of any washout. The bearing power is also usually high, and although in America piles are very frequently used with gravel foundations, they are rarely employed abroad, the masonry being begun directly on the gravel bed. There are examples in America, however, where no piles were used, as at some of the Ohio dams, and at the bridge dams at Yosts and Amsterdam on the Mohawk River, N. Y. In the two last-named instances the gravel was too compact to permit the use of piles. It is, of course, necessary to use riprap or other protection as with sand foundations where there would be danger of erosion, since a strong current will quickly carry away the loose material.

It is rarely practicable to drive wooden sheet-piles satisfactorily in gravel, as they almost always spread, and often shatter before much penetration can be obtained. Steel sheet-piles are sometimes used instead, but where the gravel is coarse they also are hard to drive, and the penetration may in consequence be considerably less than planned. Thus at the dam at Yosts on the Mohawk River, N. Y., built on gravel, piles 16 feet long could in many cases be driven only 8 feet. The objection has also been made that should rusting injure them they could not be renewed, nor would the harm become apparent until some resulting accident revealed it. Experience with their durability under water is as yet very slight. A large number were driven in the harbor of Havana, Cuba, in 1910, and when withdrawn fourteen months later the pieces were found to be without rust and in perfect condition wherever they had been imbedded in the mud bottom. Above that line they were not noticeably deteriorated, as the marine growth had assisted in protecting them. How far rusting would take place in a porous foundation, such as gravel, where the water would have free access to the steel, is not definitely known.

On account of the difficulty of obtaining satisfactory work cut-off walls are sometimes used instead of sheet-piles in gravel foundations. They should not be thinner than 4 or $4\frac{1}{2}$ feet, or the room for excavating will be insufficient. The excavation, if of any depth, is usually made between sheeting driven as described in Chapter IV, under the clause "Construction of Abutment."

Hardpan.—This term is used to indicate a nondescript material which in some cases resembles a very soft and friable rock, and in others a hard dry clay. As it usually possesses a very high bearing power some engineers allow a unit pressure on the foundations not much less than that used for rock. Hardpan, however, is very apt to be seamy, and cases are not uncommon where a bed of this material apparently satisfactory on the surface has been found later to be underlaid with seams of quicksand of considerable depth. Under such conditions a settlement of the masonry is very liable to occur unless the natural formation is such that the underlying sand cannot seep out. If, however, the hardpan is several feet in thickness and of sound quality, occasional small seams of sand in it rarely cause any trouble, unless there is a possibility of their flowing out. If this is to be apprehended a thin cut-off wall set into the material and carried to a sufficient depth below the general level of the masonry will in most cases prevent trouble. As certain classes of this material are liable to erosion under strong currents any vulnerable points should be well protected with riprap, and be watched from year to year.

Rock.—This is justly considered the best material for foundations, provided it is not too seamy. As a rule, the layers of rock at the bottom of a river are considerably thicker and denser than the layers which may be exposed along the bank above, and the greater the depth the greater is the probability that the rock will be found solid. Where small seams of foreign matter, such as sand or clay, are found under the foundation, they will not necessarily be a source of weakness if there is no danger of their flowing out or becoming a source of sliding, and in most cases adequate protection can be obtained by employing cut-off walls as described for hardpan foundations. If conditions are satisfactory and the general surface of the rock is at a suitable elevation and sufficiently uneven to prevent any movement, the masonry can be commenced directly upon it. If the rock is solid, all blasting should be avoided as far as practicable, as it usually leaves the foundation in a split and seamy condition. It should be ascertained, however, both for rock and hardpan, that no seams of foreign material or cavities exist near the surface which will be liable to give trouble later. The failure of a large reservoir dam in Pennsylvania, built on a foundation of sandstone in layers from 1 to 3 feet thick, but divided by seams of clay and shale from 2 to 4 inches thick, was due to the water penetrating from the upstream side into a seam about 5 feet below the surface. Part of the dam slid on this joint for about 18 inches, splitting and carrying its rock foundation with it and cracking the masonry. The dam was allowed to stand in this condition for more than a year, in spite of the protests of most of those concerned, but it eventually was washed away. Another example will be found in Chapter X, under the clause "Sliding."

In drilling test holes, especially in limestone, it happens not infrequently that

a spring is met with that flows up through the hole, but if the water runs clear no danger of a washout need be apprehended. If desired, channels or cavities in such seams may be filled with grout by placing pieces of pipe in the drill holes (the joints being calked with wooden wedges) pouring in the grout, and then using a close-fitting rammer in each pipe (like the plunger of a force-pump) with one or more men to force the grout downwards. This is usually effective in stopping the leaks and filling any open seams. In important or difficult cases the cavities may have to be washed out by forcing air or water through them, a grouting pump being then used to refill them.

An interesting example of the treatment of a seamy rock foundation is that of the Hale's Bar Dam, just below Chattanooga on the Tennessee River (Pl. 57). The rock was full of seams and cavities of varying sizes, to many of which the river appeared to have free sub-surface access (Fig. 144*a*). From 3 to 6 feet of the surface rock were excavated for the dam foundation and all cavities exposed thereby were cleaned out and filled with concrete. Drill holes were next put down, in some cases 4 feet apart, and all other cavities revealed by them, if not more than 10 feet below the surface, were washed out by forcing in air or water, and were then filled with grout by a grouting machine. Cavities at a greater depth than 10 feet were in general not treated, although many were found 30 feet below the surface. Some of the cavities were filled with clay, others with sand and gravel. Wherever believed advisable the masonry was anchored to the rock with 1-inch square bolts set about 3-foot centers, and extending several feet into satisfactory rock. The low-water lift of this dam is 41 feet.

Where the rock is higher than the finished surface of the masonry is to be (as sometimes is the case with the foundation for a movable dam) and is of good quality, a thin base is often used set into a trench in the rock and bolted thereto. Thus part of the sill masonry of the bridge dam at Fort Plain, on the Mohawk River, N. Y., consists of a bed of concrete only 3 or 4 feet in thickness and set into the bed-rock. Similarly part of the sill masonry of the Chanoine wicket weir of Dam No. 2, Big Sandy River, is of shallow concrete bolted to the rock, and part of Dam No, 18 on the Ohio River is similarly built. (See also "Walls in Rock," p. 404 and after.) In such cases it must of course be ascertained beyond all question that there are no seams in the rock which might be a cause of sliding, as in the examples just referred to.

Leaks and Springs.—(See also "Undermining" in Chapter X.) More or less trouble is usually experienced during construction, especially with gravel foundations, from leaks and springs. Sometimes the leakage can be reduced considerably by dumping earth just outside the cofferdam. If a special leak develops and its channel can be reached, it can often be stopped by letting the water rise until quiescent and then filling the channel with clay rammed through a pipe. Whatever water comes through the cofferdam usually has to be led to one side in a

trough or ditch and be carried to the sump or pit in which the suction of the pumps is placed. This sump should always be located so that it will not have to be interfered with until the work in the cofferdam has been finished, and also that it will be where all drainage can flow to it by gravity. (See also the following clause, "General Notes on Construction.")

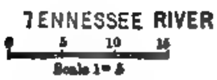


FIG. 144a.

Those leaks or springs which appear under the site for the masonry and which are due sometimes to leakage through the cofferdam and sometimes to natural springs, are more difficult to deal with. In some cases they can be controlled by setting an open box above them and filling around it rapidly with concrete; the water will then rise in the box to a point where its own weight will check the

flow and the box can be filled later with concrete. Another method is to build in a pipe and to lead the water through it to one of the nearest ditches. The pipe should be provided with a connection near the face of the masonry and when the proper time arrives the outer portion of the pipe can be removed and a wooden plug be driven in and the whole be covered with concrete. Where the springs are small they can often be effectually checked by dumping concrete directly over them, and in many cases this will obtain a sufficient set before the water can break through it. If it does not, the escape of the water through the concrete will do no permanent harm, and very often the chemical conditions which slowly generate result in finally sealing the outlet. The closing of a spring often tends to make it break out elsewhere in the vicinity, but with a gravel foundation this is usually of little consequence, since it does not produce undermining. With clay or sandy material, however, the water will sometimes remove a considerable amount of material, and in many cases this can be filled with gravel in a manner that will let the water escape and at the same time stop erosion.

One of the most difficult foundations on which a dam has been built was that of the Assiout Dam on the Nile, where the natural foundation consisted of a very fine sand and silt. Springs were met with over the entire area, often breaking out without any warning and some of them so large that they filled an 8-inch pipe. These were usually checked by building sand-bag cofferdams around them and allowing the water to rise until quiescent, when the outlets were filled with concrete. The small springs often developed into large springs, and much of the concrete of the floor had to be laid in small sections with vent holes every few yards. Iron pipes 4 inches in diameter were used, built in with the concrete and a few days later other lengths of pipe were screwed on, reaching a height of 18 or 20 feet. These were later filled with grout, thus closing all cavities. In the case of some of the largest springs the sand around them was eroded in a short time into holes with a diameter of 8 or 9 feet.*

General Notes on Construction.—In beginning any construction which will restrict temporarily or permanently the flow of a river, and thus produce swifter currents and a tendency to erosion, the shore end should be properly protected early in the work. Thus with a cofferdam or a dike, especially if on a concave bank, the junction with the bank should be well riprapped or otherwise protected to prevent the water flanking or cutting behind it in floods. This work should not be left until the construction is finished, since floods often come unexpectedly, but should be done as soon as practicable.† Similarly, in building a lock and dam all permanent work which may be necessary to secure both banks before the river is contracted to any effective extent by the cofferdams should be done as

* Proceedings Institution of Civil Engineers, vol. clviii, 1903–1904.

† For general remarks on cofferdams, see p. 348, and after.

early as practicable. Cases have occurred where this precaution was neglected, and the contraction of the river caused erosion of the unprotected abutment bank to an extent which necessitated a considerable extension of the dam.

Where the excavation for the lock or the abutment is commenced by dredging, restriction should in most cases be placed on the depth and the amount. This is especially necessary with high alluvial banks. These are usually underlaid with sand, and as soon as the dredge has cut out the supporting toe the weight of the bank and the flow of the ground water begin to force out the underlying strata and the bank cracks, settles and slides. If the cut is exposed to currents the sand is carried away and the bank will continue to settle indefinitely. In one case where a dredge working for a few weeks cut too far under a bank, the earth continued to slide for six months, and until the break had extended nearly 200 feet from the original bank line, the caving ceasing only when the foundation of the lock wall had been built up to the seepage line. The cost and delay in removing the sliding material and in placing the additional backfilling was very considerable. Where protected from currents by a cofferdam the sliding is less pronounced, but even in such cases it may become extensive unless the dredging has been limited.

On most rivers in America the excavated material is of a nature which can be dumped into the adjoining natural channel and will be carried away by floods, giving no further trouble. Where, however, the river or the material is of a character which is unsuited for such treatment the excavation must be disposed of on the bank or where it will not disturb the regimen of the stream.

The seeping-out of the toe of a bank during excavation and the resulting continued sliding of the earth is one of the most troublesome features met with in lock and dam construction. There is a certain seepage or ground-water line reached in the excavation above which the material remains stable and below which it will begin to flow out and cause a sliding of the bank, and the further down the excavation is carried the worse the conditions become. Wherever practicable—even at an increase of expense—the general excavation of the bank should be stopped when this seepage line is reached and the foundation of the walls should be put in in short sections (using sheathing as necessary), so that only a short length of bank will be exposed to caving at any one time. As soon as the masonry is built up sufficiently the caving will stop and the bank behind it will become stable again. For this reason, as well as to protect the bank against floods as before described, it is usually desirable to complete all the foundation masonry on the bank side as soon as practicable, leaving the upper portions to be built later.

After a cofferdam has been kept pumped out for a short time the seepage line will usually become lowered. If the cofferdam becomes flooded the bank will become

saturated again, and if it is of a treacherous nature the cofferdam should be pumped out slowly, or the material will begin to break and slide just as river banks do when a flood recedes quickly.

A general scheme for draining the site of the work should be carefully planned beforehand. The sump or drainage pit for the suction pipes of the pumps should be placed in the lowest spot, so that the water from springs and leaks will flow towards it naturally. It should also be placed where it will remain undisturbed, as any shifting of pumps leads to considerable delay and expense. Having decided on the location of the sump, the work should be planned and conducted so that the water will always have a free outlet as far as practicable; that is, no portion of the structure should be built which will cut off the escape of water from another portion not yet built and so necessitate special or additional pumping. The general scheme of drainage and the sequence of construction often require very careful study, and if it can be planned so that the leakage will always have uninterrupted flow toward the sump it will greatly facilitate progress.

It is usually best to complete as early as practicable the parts most exposed to interference from leakage, such as the lowest portion of the foundations, etc. These parts, if not secured or completed early in the work, will be a source of delay and expense as long as they remain unfinished. When, however, they have been built up to a suitable height above the water level, not only will the cost of pumping and hindrance from water be reduced to a minimum, but should the cofferdam become flooded, the expense of re-excavating for the deep foundations and of protracted pumping will be avoided. With a satisfactory cofferdam, a good system of drainage and reliable pumps, with methods which will allow the early construction of these parts where caving and leakage would give most trouble and will provide for as little rehandling as possible of excavation and materials, the difficulties and delays inherent in work subject to interference from rivers will be reduced to a minimum.

Backfilling.—Backfilling or placing material behind the lock walls, abutment, etc., is usually done with the earth, etc., excavated during construction, no special attempt being made to separate coarse and fine materials. This backfill acts as an earth dam in separating the upper and lower pools, and where the distance is short, as behind an abutment, care should be taken to avoid the collection of sand or gravel in layers which might lead to seepage. A good loam forms an excellent backfill for such situations, and if dry enough it should be compacted; if wet, it becomes dense enough when settling. Behind a lock wall no special selection of material need be made, except at the upper wing or cut-off wall, nor is it necessary to compact the fill except for use with paving, and where the material can be allowed to settle, say for a year, the paving can usually be laid upon it without compacting being necessary. The distance from pool to pool in these situations is considerable and appears to be ample in actual practice to safeguard the structure without the precaution of compacting.

If the backfill is wet, it should be placed a little at a time, to avoid undue pressure on the wall. A bridge abutment, being built by a railroad company at a crossing of the New York State Barge Canal, was destroyed from neglect of this precaution. The wall was of heavy section, carrying a skew 6-track crossing and a considerable surcharge of embankment, and rested on compact gravel without piles. The backfill was made by dumping semi-liquid material from an orange-peel bucket, each bucketful tending as it fell to compress and wedge out the fill already in place. As the result the wall gave way some time before the filling was completed, breaking in several places and leaning over till it was 5° or 10° out of plumb.

Rock Excavation.—Where a wall is to be built against an excavated face of rock, the drill holes when blasting should be set about a foot apart in order to secure a good line and avoid shattering the rock mass. If spaced several feet apart, as is often done, the result is a ragged line and usually much additional excavation and masonry, as the rock breaks some distance back from the holes, and even if not shattered enough to become loose it is generally badly cracked. Channeling is sometimes resorted to for avoiding this trouble, but unless a large amount is to be done it does not pay to provide the machine, and the method of close-spacing the drill holes is in most cases cheaper and equally satisfactory. The specifications should, however, describe clearly any special drilling of this nature, as it naturally costs more than the method usually employed.

It is also desirable at times to excavate test-pits in rock. In such cases the first surface holes should be inclined towards each other at about 45 degrees to the horizontal, so that the blast will tend to expend itself upwards in blowing out the core. This will leave a bowl-shaped cavity and provide a face for the next blast, the holes for which can be put down vertically. By continuing this method, which is the same as is used in tunneling, the shattering of the rock adjacent to the test pit can be avoided.

Materials of Construction.—Modern locks and dams, and the foundations of movable dams are almost always built of concrete or stone masonry. Where placed in a derivation as at Herbrum on the River Ems, the lock has sometimes been formed by simply building masonry ends to the chamber, and grading and paving the banks between so as to form the lock-pit. Fender piles are generally used to keep the boats off the paving. A few examples of wooden locks of modern construction are to be found on the Fox River in Wisconsin, where the extreme flood range is only about 3 feet, and serve their purpose excellently. They have masonry ends, and the chamber is formed by upright posts covered with two thicknesses of 2-inch dressed plank, jointed but not calked, behind which a dry rubble wall is built. The cost of these locks is about one-third that of masonry locks, but they have to be renewed about every ten years. Locks built of crib-work faced with plank and filled with stone or gravel, with cut-offs at each gate, were used for much of the early construction in America, and similar structures are still used in Russia. For locks exposed to high floods, however, masonry

should always be used, as its mass is necessary for endurance. The choice of materials for it is limited usually to sandstone, limestone, or concrete. Limestone is usually preferable to sandstone, as it does not soften and wear so easily under the action of the water.* Concrete is almost universally used at the present time in the United States, although experience there with its durability in river works has been comparatively short. There seems to be no reason, however, why it should not prove satisfactory, and in fact concrete locks which had been built in 1795 on the Francis-Joseph Canal in Hungary, were still in use in 1900. While somewhat battered in appearance, they were generally in sound condition, and probably no worse for wear than cut stone locks would have been. For works exposed to the action of a river it is now customary to use a concrete entirely of Portland cement. The difference in the cost of Portland cement and of the natural cements once in common use became small, and the former gave a harder and better concrete in all particulars. The higher grades of natural cement, formerly widely manufactured when Portland cement was costly, have given excellent results in foundations, but the cheap grades have not proved satisfactory.

Masonry of one-man rubble or larger, and either coursed or uncoursed, is largely employed in Europe. If the face of the lock chamber is of rough stone, vertical fenders of cast or wrought iron are set against it. The use of concrete in Continental practice has been largely confined to parts under water, but within the last few years entire structers have been built with it. In Egypt are locks composed of rubble walls consolidated by pouring in grout. Examples are found in several countries of walls of cut stone with rubble backing, and occasionally with concrete backing. Brick has been used within recent years on the Nile, and many older examples of brick lock-walls are to be seen in Europe. Fenders are usually placed on them to prevent abrasion.

The use of timber for permanent construction, wherever it will be exposed to the action of moving water and cannot be easily replaced, should never be permitted. The accepted belief that timber below water lasts forever is only partly true, for in order to do so it must be protected from all currents. This fact was discovered long ago by European engineers, but in America there are still to be found many examples of wooden lock floors, wooden foundations, and other vital parts which the water will sooner or later wear away. The sheathing of a dam bears strong witness to the rapid wearing force of water; and while a lock floor is exposed to much slighter currents, the effect is just as sure. We have seen wooden lift-walls, exposed only to leakage from the gates, become dangerously worn in twenty years, and have removed timbers from the inside of a dam, where exposed to leaks, which were worn and channeled half away. Wherever it can possibly be done, nothing but masonry should be exposed to

* The softer classes of limestone, especially the oölitic varieties, have not proved durable in the face work of lock masonry, as after a few years' exposure to the weather the stones begin to scale off. For a similar reason no machine-finished surface and no bush-hammered surface should be used in exposed work, as the action of the tools deadens the surface and causes it to scale.

the action of the water, especially as timberwork has now become almost as expensive as the more permanent materials.

To quote an example of the ultimate risk involved, a dam on the Spree was removed in 1890 after an existence of many years. The original foundation had been composed of an upstream and a downstream row of sheet-piling with round piles between, carrying a platform of square timbers on which the masonry rested. The head of water had rarely been more than 3 feet. On the removal of the dam, it was found that the seepage of water under the foundation had eaten out much of the sheet-piling along the joints and had worn deeply into the under sides of the platform timber. Water channels were found running under the work in many cases to a depth of 18 inches. The engineer in charge of the removal stated that in a few years more the supporting woodwork would have been practically destroyed.* On the Welland Canal in Canada the wooden planking of the lock-chamber floors, exposed only to the occasional currents from the filling valves in the lock gates, was found after some twenty-five years' use to have been almost worn through. Where the water carries sand, the wearing of timber exposed to its action is often very rapid, as on a dam on the Brantas River in Java, where the wooden planking of the foundation had to be frequently repaired, and finally protected by a sheathing of cast iron.

The use of iron or steel under water or where it cannot be replaced easily should always be avoided as far as practicable. Wrought and cast iron, however, appear to be affected but slightly by river water in its natural state, cast iron being especially durable. Wrought steel, on the other hand, rusts very quickly, especially if there is any trace of acid or of sulphur in the water. Nickel steel is said to have more durability, and has been used for lock gates on the Monongahela River, where in low water the corrosive action is extremely rapid. Experience with cast steel under water appears to be conflicting, and because of this it is preferable to use another material whenever practicable, such as cast or wrought iron, whose durability has been proved. In special cases, as for important bolts and fittings, bronze has sometimes been used.

Protection of Banks.—This will be found described in Chapter IV under "Fixed Dams—Abutments."

Clearing the Pool.—After the location has been selected, the river above should be cleared of bowlders, snags, and timber throughout that part which will be affected by the new pool. It is best to commence this work at once, so that debris will disappear before the dam is finished, and also in order that steamboats may be able to utilize the stream in moderate stages of water as soon as the obstructions have been removed.

If it is not expected to complete the lock and dam for some years, the trees

* Proceedings, International Congress of Navigation, 1898.

can be merely deadened, or "ringed." When this is done, the trees will gradually die and fall, a branch at a time, into the river, and will be safely carried out by the floods. Some trees, depending on size and species, will die and begin to disappear in eighteen months, while others, especially sycamores, will struggle for life from three to six years. Where they are below the level of the new pool, they should be deadened close to the roots, so that no projecting stump will be left when the tree falls.

If it is desired to remove the timber at once, the trees must be deprived of their branches and the trunks be cut into lengths of 10 or 15 feet, so that they will pass out of the river without forming snags or injuring craft. As with deadened timber, they should be cut off close to the ground, so that there will be no submerged obstructions left. Above the level of the new pool it will only be necessary to remove the timber that overhangs the river, and in wide streams even this is not required. Where the trees are large, cross-cut saws are more economical for their removal than axes. Dangerous stumps, if the ground is hard, can be removed by Judson powder, a mixture of black powder and dynamite, and which must always be fired with exploders or percussion-caps. It is very useful also in removing broken or creviced rock, as it can be poured into the cracks, thus saving drilling. All dangerous bowlders or reefs should be blasted out, if possible to several feet below the depth required for navigation. For this it is best to use a high grade of dynamite, as it will shatter the rock more and make its removal easier.

The cost of clearing a pool, where the timber is heavy and reefs and bowlders plentiful, may reach as high as $800 or even more per mile. Additional information as to methods and costs will be found on pp. 125 to 132.

General Remarks on Design.—In designing the various parts of a lock and dam, the engineer should bear in mind that repairs to works of this nature are liable to be costly and tedious, not only because the presence of water has to be contended with, but also because the lock or dam may have to be placed out of operation, thus interrupting navigation. The effect of such a stoppage, if the commerce is large, is somewhat similar to the inconvenience of a tie-up of the freight traffic of a railway. For this reason especial attention should be given to making all parts as simple as possible, and of ample strength in those places where they may be subjected to blows from boats or other mishaps, or to special wear. In addition, if the parts are connected with the operation, such as valves, pintles, etc., they should be so designed that they can be easily removed and replaced when worn or broken, and, as far as practicable, without having to interfere with navigation or pump out the lock. This is a matter which is unfortunately too often overlooked, and forms one of the unnecessary and expensive difficulties to be contended with in repairs to existing structures.

The same principles apply to movable dams with still more importance. In the design of works of this class, where much material of temporary life must be employed, such as wood or iron, a large excess of strength should be provided, not only because wear and rust will weaken them, but because they are subject to blows and twisting from drift and ice, and occasionally from boats, the forces of which must always remain indeterminate. As far as experience in this country has shown, the life of the trestles and wickets of a Chanoine dam appears to be about twenty years. In that time the water will practically wear out the timber of the wickets and will eat up the ironwork of the trestles, and the latter process appears considerably more active on steel than on iron. It has not been found practicable to protect such iron work adequately with paint, as the scour of sediment and the long immersion soon damage and undermine the coating, and once an entrance has been gained, the rusting will continue beneath slowly and insidiously, and will create a surface so rough that it can never be thoroughly protected again. The short life of the metalwork is supposed to be due, in part at least, to the fact that on those rivers where rusting is most noticeable the water contains more or less sulphur or other chemicals, derived from the discharge of mill waste into the river or from the natural drainage of the coal districts through which many of them pass. Where the water is comparatively pure, and the metalwork is of iron instead of steel, the life of the pieces is much greater. Thus the trestles and wickets of La Mulatière dam at the mouth of the Saône were in excellent condition after twenty-five years' service, and the trestles and drums of the Marne were only slightly less deteriorated after a use of forty years.

All journals, pins, or other movable parts which are to be subject to immersion should have a play of at least $\frac{1}{32}$ of an inch in their holes, or they will become so rusted in a few months that their removal, or operation where they act as hinges, will be very difficult.

Provision should be made for renewing the anchor-bolts of lock and dam-sills, etc., since their exposed ends will sooner or later rust away unless they can be protected by mortar or similar means. This provision may be made by putting sleeve-nuts or turnbuckles on them near the top. It will then be necessary for renewal only to excavate to the turnbuckle and to screw in a new end for the bolt.

Where several locks and dams are to be built, a uniform design should be adopted as far as practicable for all similar parts, so that the machinery of one lock will be interchangeable with that of another. This facilitates not only the work of construction, but also the work of repairing any parts that become worn or broken. This applies to the lock gates also where the traffic may be heavy, as they are liable to be injured or destroyed by collisions. (See Chapter X.)

After the works have been completed all débris should be removed and the grounds should be laid out, graded, and planted with grass and trees. Lock-tenders,

with a little encouragement, will take pride in keeping their premises trim and in good order, and although it may require some extra expense to secure these results, it will be found that it will be very small, and hardly worth considering in view of the general advantages gained. Moreover, in works of such magnitude and under the charge of the Government, it may justly be claimed that they should be completed and cared for in all respects with a high degree of excellence.

Silting-up of Pools.—(See also paragraph "Silt in Reservoirs," p. 301.) When an obstruction such as a fixed dam is placed in a river in a manner that will cause a slackening of the flow, it produces more or less tendency for deposits to form wherever the velocity of the water has been checked. Where the sediment is very coarse, such as gravel, these deposits usually form from year to year, and have to be removed by dredging. Instances of them may be seen just below mouths of steep tributaries, in which localities are often found beds of the gravel which the streams have deposited in the comparatively still water of the main river. However, the sediment transported in those portions of rivers which are slackwatered is as a rule of a much finer character, and as far as experience goes, very little trouble results from the slackening of flow caused by a fixed dam. On the Green River in Kentucky, which was slackwatered about 1840 and which carries a large amount of sediment, there are stretches of the river where the channel has constantly maintained low-water depths of 40 feet and more, and no signs of any tendency for the bed to silt up have become apparent. Soundings taken about 1890 on the river Severn in England and continued for a distance of about 1½ miles above each of the fixed weirs show, by comparison with soundings taken in 1842 (before any weirs had been constructed), that the channel had deepened from 3 to 5 feet, while other soundings appeared to show a small increase of depth through all the pools.* On the other hand it has been stated that the construction of the movable dams on the lower Nile, where the volume of silt is very large and is deposited by a very slight reduction of velocity, there has been apparent some tendency for portions of the bed to silt up, although the river so far appears to remove these deposits when the dams are open during the seasons of high floods. With movable dams the tendency to cause permanent deposits is as a rule too small to have any practical effect, as the river is given a free channel when the dams are not in place.

It is believed by many engineers that the small effect which fixed dams appear to have as regards causing deposits is largely due to the fact that their obstruction during floods, when the river is carrying the greatest amount of sediment, reduces the total area of discharge only by a small amount.

Saturation of Land.—When the normal level of a river is raised until it stands only a short distance below the surface of the adjoining fields, it tends to change

* Proceedings Inst. C. E., vol. lx, p. 77.

the ground-water level to an extent which may affect the neighboring fields. Experience has shown that in light soils the surface of the pool can stand about $2\frac{1}{2}$ feet, and in heavy soils about 3 feet, below the general level of the fields without causing injury. Small floods, provided they do not overtop the banks, do not affect this limit, as they usually pass off without affecting the ground-water level for more than a short time.

This question is one of importance in canalizing rivers with low banks and flat slope, where the lifts must be small even under the best conditions. Every foot that can be added to the lifts means so much deeper water for navigation. As explained further on in the chapter on "Movable Dams," the pool level can be made higher with this type than with a fixed dam, and for some distance just above a movable dam the limit of $2\frac{1}{2}$ to 3 feet, mentioned above, can sometimes be reduced considerably if intercepting ditches are dug near and parallel with the banks and connected with the lower water level below the dam. These ditches serve the same purpose as the drains or ditches dug along the outsides of canal embankments to intercept seepage water, but the method is only effective within moderate limits. Thus in certain cases where pool levels already existing have had to be raised to meet the demands for greater depth, dikes have been built accompanied by ditches as just described, but even with these precautions some of the adjoining land has become permanently injured.* In such cases the value of the land must of course be balanced against the saving effected by not having to deepen the river bed.

COST OF CANALIZATION.†

Black Warrior and Tombigbee Rivers, Alabama.—Canalized with fixed dams. Cost about $19,000 per mile. Least channel depth, 6 feet.

Hudson River (New York State Barge Canal).—From Waterford to Fort Miller, about 30 miles, cost about $187,000 per mile.† The distance included 5 locks with a total lift of 87 feet, one new fixed dam, 3 lateral canals from $\frac{1}{2}$ mile to $1\frac{1}{4}$ miles in length, and a large amount of dredged channel of 12 feet navigable depth and 130 to 200 feet in width. Much of the channel was excavated in solid rock, some of the cuts ranging to 18 feet in depth. Four existing dams (all of fixed type) were utilized as a part of the improvement, and their cost is not included. This is probably the most costly river improvement yet undertaken, considering the short distance.

Muskingum River, Ohio.—The original construction of 9 locks with fixed dams, canalizing 80 miles, cost about $16,000 per mile. Least channel depth thus obtained was $4\frac{1}{2}$ or 5 feet.

* Seventh International Congress of Navigation, 1898. Paper by E. Roloff.

† Further details of several of the systems mentioned will be found in the Tables of Locks and Dams at the end of the book.

Big Sandy River, W. Va. and Kentucky.—Canalized with 5 locks with movable dams (needles and Chanoine wickets), giving slackwater for about 50 miles. Cost about $28,000 per mile. Least channel depth, 6 feet. Average length of dams about 260 feet.

Kanawha River, W. Va.—Canalized with 10 locks, 2 fixed dams, and 8 movable dams (Chanoine wickets) giving slackwater for about 90 miles. Cost about $40,000 per mile. Least channel depth, 6 feet. Average length of the movable dams, about 550 feet.

Mohawk River (New York State Barge Canal).—From Cohoes to Mindenville, about 69 miles. Cost about $138,000 per mile.* This included 9 locks with a total lift of 118 feet, 2 large concrete dams, 8 movable dams of the bridge type with an aggregate net length of 3750 feet, and about 8,000,000 cubic yards of dredging, chiefly gravel. The least channel dimensions in the open river were made 200 feet in width and 12 feet in depth.

Ohio River.—Canalization now under way, using Chanoine wicket dams with bear-trap drift-chutes. Estimated cost per mile for a least channel depth of 9 feet, about $75,000. The dams are very long, the shortest being about 900 feet; the longest one may be two or three times this length.

Fulda (Germany).—Cost per mile about $52,000.†

Main (Germany).—Cost per mile about $103,000.† (About $5\frac{1}{2}$ feet depth.)

Oder (Germany).—Cost per mile about $147,000.† (Estimated, 5 feet depth.)

Elbe-Moldau (Bohemia).—Estimated cost of canalization with double locks and movable dams, about $23,000 per mile, Channel depth 6 to 7 feet.

Seine (France).—From Paris to tide-water, about 140 miles with a total fall of $83\frac{1}{2}$ feet, cost for the first improvement (1838–1853) about $2,800,000 or about $20,000 per mile, the channel depth being $5\frac{1}{4}$ feet. The next improvement (1858–1878) cost the same and gave a channel depth of $6\frac{1}{2}$ feet. The last improvement (1878–1888) cost $12,200,000, or about $87,000 per mile, and gave a channel depth of $10\frac{1}{2}$ feet. This system includes 9 locks with movable dams. Total of all costs $17,800,000, or about $127,000 per mile.

* Cost of construction only; cost of surveys, engineering, etc., are not included.

† Proceedings International Navigation Congress, 1912.

CHAPTER II.

LOCKS.

Origin.—The date of the invention of locks is somewhat uncertain. By some they are ascribed to the Dutch, and are said to have been originated in 1253; by others they are claimed for Leonardo da Vinci, while still others say that Philip Visconti was the inventor. In the "Annales des Ponts et Chaussées" for 1847 it is stated that the first one was built to facilitate the transport of marble for the Milan cathedral, and Lombardini, an Italian engineer, says this was done by Visconti in 1439 to connect the old and new lakes, the difference of level being about 10 feet, and he claims further that da Vinci could not have made use of locks until 1460.*

Description.—A lock consists of a rectangular basin called "the chamber," having side walls of masonry, earth or timber, connected near the ends by gates of wood or metal. Through the chamber thus formed communication is established between two pools of different levels by admitting water from the upper pool through conduits and discharging it into the lower pool at the lower end. That part of the lock above the upper gates is called the head bay or fore bay, and is flanked by the head-bay walls, and that below the lower gates the tail bay, flanked by the tail-bay walls. (See Fig. 145.) Between the tail-bay walls is the lower coffer-wall, used for coffering the chamber. The gates close at the bottom against sills inclined upstream and called, from their position and construction, the upper and lower miter sills. The lower miter sill is generally attached to a wall connecting the side walls of the lock and called the lower miter wall, while the upper miter sill is attached to the lift or upper miter wall; this wall also connects the main walls of the lock and is sometimes called the breast wall. The upper miter wall frequently contains the filling culverts. The head-bay walls are connected by a cross-wall, sometimes also called the breast wall, but generally known as the upper

NOTE.—Certain of the illustrations in this and the following chapters are reprinted by permission from "The United States' Public Works' Guide and Register," by Captain W. M. Black, Corps of Engineers, U.S.A.

Table giving sizes of locks, etc., in America and elsewhere will be found at the end of the book, and Pl. 46 shows the general location of the principal canalized rivers in the United States.

* The first lock on the American continent was built at Sault Ste. Marie, Canada, in the year 1790. The lock was 38 feet long, 8 feet wide, with a lift of 9 feet and a draft of 2½ feet. It was used by the fur-traders for lifting their heavily laden birch-bark canoes to the level of Lake Superior, but was destroyed in 1814 by the U. S. troops. The old sills and floor timbers, however, are still in existence, and some years ago the walls were rebuilt, and the structure is now maintained as a curiosity.

coffer wall. Of the two side walls the inner one is known as the land wall and the outer as the river wall. The land wall generally has wings at its ends extending into the bank. Gate recesses are formed in each wall just above the miter sills, into which the gates swing when opened, out of the way of passing craft. The recesses are terminated at the lower ends by masonry or castings of special construction, known as hollow quoins, into the hollow of which the gate fits when shut. At the upstream end of the recess is a right-angle, round or beveled on its outer face, called a square quoin.

Maneuvers.—Boats are let into the lock when the water therein is at the level of the pool from which they approach. To pass a boat through a lock upstream, the boat enters the lock, the lower gates are closed, and the water is let in and fills the chamber, lifting the boat to the level of the upper pool. The upper gates are then opened and the boat proceeds. If a boat desires to pass

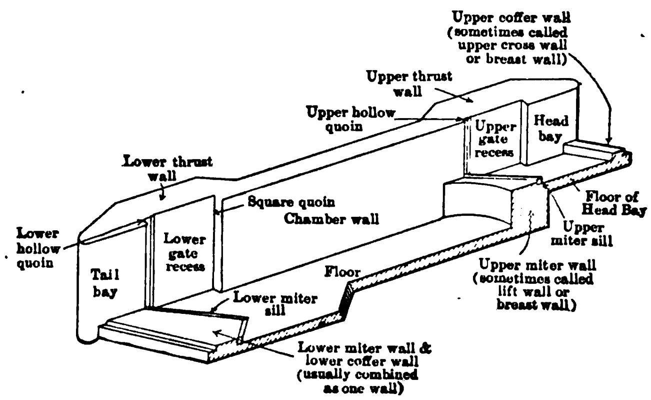


FIG. 145.—Isometric Section of a Lock. (See also p. 395.)

downstream, the lock being full to the upper level and the upper gates being open, the boat enters, the upper gates are closed, and the water is let out of the chamber till it is at lower pool level; the lower gates are then opened and the boat can go out.

Calculations for Lock Walls. *General Dimensions.*—In determining the proportions of lock walls, there are usually three main parts which govern the general dimensions. These are:

(1) The river wall of the chamber.

(2) The land wall of the chamber.

(3) The mass required in the head-bay and tail-bay walls.

There are minor portions which must be examined, such as the wing walls, which frequently act as retaining walls, the miter walls, which are usually made in arch form, the upper coffer wall or wall across the head of the lock, etc.

To calculate the stability of the walls, certain preliminary assumptions must be made so as to obtain the volume. The width of coping AB (Fig. 147) of the

chamber walls as usually built is from 5 to 7 feet. For a river wall a width of 5 feet is somewhat narrow, but one of 6 feet is usually sufficient for all moderate lifts, and 7 feet for high lifts, while for a land wall, if filled with earth level with its top, the width can be made 5 feet or less, 3 feet having been used for the lock on the Mississippi River at Moline, Ill., and one meter ($3\frac{1}{4}$ feet) being a common width in Europe. The batter *BC* usually runs about $3\frac{1}{2}$ inches or 4 inches to 1 foot for a rock foundation, and 4 inches or $4\frac{1}{2}$ inches to 1 foot for a porous foundation, and the base *CD* usually varies between 45 and 65 per cent of the height, steps being used to obtain the required base width. (See sections of walls on Pl. 45*a*.) The distance *AE* (known as the "guard") can be assumed as 3 feet for a movable dam, and 6 to 12 feet for a fixed dam, depending in the latter case on the depth expected on the crest where the range of flood is small, and on the stage at which navigation is expected to be suspended where the range is large. (See also "Height of Lock Walls," p. 343.) Where fixed dams are provided and the floods do not rise over the lock walls, the tops of the walls are usually placed from 1 to 2 feet above the flood level, 18 inches being frequently used. Where floods rise over the walls (as is the case on most of the southern and western rivers of the United States where the winter floods in extreme cases have submerged certain locks to a depth of 15 to 20 feet) the tops of the walls should be placed if practicable at such an elevation that just before the lock begins to be submerged the fall over the dam disappears, and boats can pass up or down in the open river. This cannot always be done, however, especially with rivers of high floods and in narrow valleys, and in such cases the walls are usually built 10 or 12 feet above upper pool level. When the floods have risen above that height; the current is often too swift for safe navigation. (See also "Fixed Dams and Flood Levels," at the end of Chapter IV.)

The width behind the gate recess (*AB* or *FG*, Fig. 153) should not be less than 9 feet, but need not be more than 12 feet, unless for some special purpose. The additional width as compared with the chamber coping is usually needed only for maneuvering the gates, and not for the support of the gate spar, which can and often does project many feet behind the wall when the gate is open. The rear faces of these walls are usually battered 1 inch to 2 inches per foot or more as required, and the bases widened as needed by steps.

The least width of the upper miter-wall coping is usually taken as 5 or 6 feet, part of this being upstream of the sill.

River Wall of Chamber.—The normal forces affecting the river wall of the lock chamber are the weight of the masonry and the pressures from the pools; the extreme forces may result from the chamber being full with the lower pool drawn off or reduced, or from a combination of flood levels which, while lessening considerably the low-water head, nevertheless bring the pressures to a higher point

MODEL OF THE LOCKS AT ST. MARY'S FALLS CANAL, MICHIGAN.

The small lock, known as the Weitzel lock, was finished in 1881, and the large one, known as the Poe lock, in 1896. A portion of the floor of the latter is removed to show the culverts. The small lock is 80 feet wide and 515 feet between hollow quoins, with 16 feet on miter-sills; the large one is 100 feet wide and 800 feet between hollow quoins, with 21 feet on miter-sills. The normal lift varies between about 18 and 20 feet.

upon the wall and thus increase the tendency to overturning. The condition must also be investigated of having the lock pumped out during construction or for subsequent repairs at times when the pool levels are a few feet above their normal heights. The conditions of maximum loading can only be determined by trial, and will vary with the locality and the design, depending upon the probable flood elevations during operation, the type of dam (fixed or movable), and similar factors.*

With porous foundations and with the dam at the lower end of the lock or so placed that the pressure of the upper pool comes on the outside of and underneath the walls, the effect of underpressure reduces the stability greatly, and may require a very heavy section to be used. Thus for certain of the Mohawk River locks (New York State Barge Canal) built on gravel and with movable

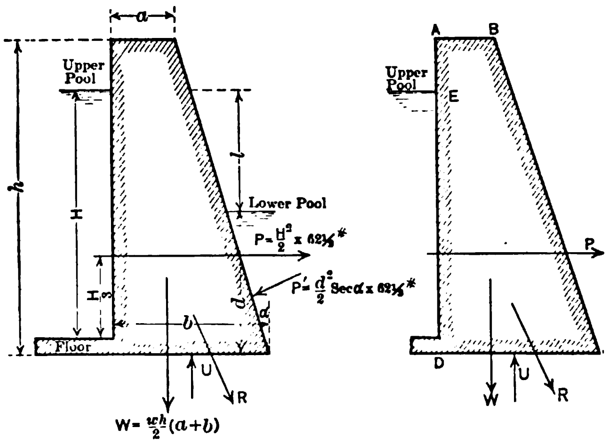


FIG. 146.—Normal Conditions. FIG. 147.—Conditions with Lower Pool drawn off.

Forces acting on a River Wall, with the Dam at the Head of the Lock.

dams placed opposite the centers of the chambers, the section of the river wall above the dam had to be made of a uniform thickness of 20 feet from the top to the floor level, where offsets were used to widen the base. The lift in these cases was 15 feet, and the total height of the wall 34 feet, the normal upper pool level being 4 feet below the coping.

It is customary to neglect the effect on the lock walls of any pressure transmitted from the floor; the walls and floor are usually considered as being inde-

* Sections of chamber walls of existing locks are given on Pl. 45a. The following rule was applied to the design of certain lock walls of moderate height and recent construction: The horizontal width through any point of the section should not be less than 50 per cent of the height of the wall above that point, and for land walls the area of cross-section should be not less than that of a rectangle whose base is 35 per cent of the height, nor greater than that of a rectangle whose base is 41 per cent of the height. These rules were applied to the parts of the walls above the level of the tops of the piles (when pile foundations were used), or above the general base for walls on rock, but were not applied to the lower parts of walls on rock, when it was assumed that the face pressures largely balanced the pressures on the backs.

pendent, although with arched floors not supported on piles there will be a thrust against the base of the walls which should be investigated, as if considerable in amount it will tend to cause both overturning and sliding.

Let Figs. 146 to 149 represent sections of a river chamber wall where U indicates the total underpressure per foot run of wall as described on pp. 363 and 364, w the weight of a cubic foot of masonry, W the total weight of the wall, and P and P' the water pressures, the remaining nomenclature being as shown.

If the dam is at the head of the lock or above the section under consideration (Figs. 146 and 147) the forces acting under normal conditions are the weight W of the wall, the pressure P on the inside face from the upper pool, the pressure P' on the outside face from the lower pool, and the underpressure U on the base of the wall. The forces for flood conditions are similar except as varied by the change of pool levels. For the condition with the lower pool reduced or drawn

Fig. 148.—Normal Conditions.

Fig. 149.—Conditions with Lock Unwatered.

Forces acting or a River Wall, with the Dam at the Foot of the Lock.

off, the outside pressure P' is reduced proportionately. With the dam at the lower end of the lock or below the section under consideration (Figs. 148 and 149), the forces are generally similar to those for the preceding conditions, except that the pressure on the outside of the wall comes from the upper pool and that on the inside from the lower pool; U also changes as described on p. 363 and after. The position of the final resultant R can be found analytically or graphically, the latter being the simpler method and one which usually gives sufficiently accurate results. The width of base may be determined by the principles given on p. 368, and if the assumed width is found insufficient, offsets can usually be added as required.

Land Wall of Chamber.—The forces acting on the land wall under normal conditions are (see Fig. 150) the weight W of the wall, the pressure P of the upper pool when the chamber is full, the underpressure U (see p. 363), and

the thrust T of the backing (more or less saturated) as described on pp. 364 and 370. The wall should also be investigated with the chamber full to lower pool level only. T represents the resultant of the dry and of the saturated earth,

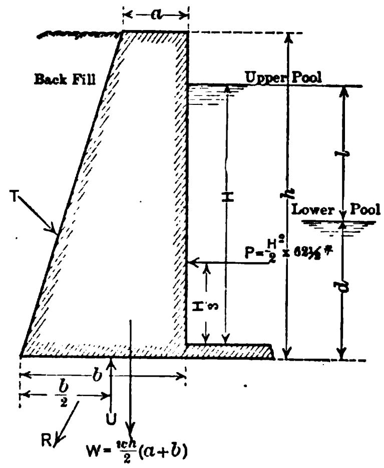


FIG. 150.—Normal Conditions.

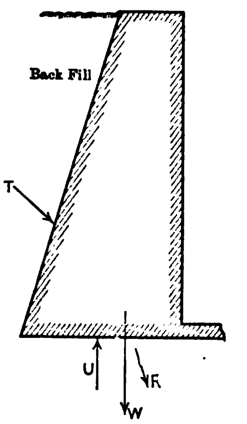


FIG. 151.—Conditions with Lock Unwatered.

Forces acting on a Land Wall.

the limits of saturation being usually taken as the same whether the dam is at the head or at the foot of the lock. The pressure of the lower pool on the back of the wall is usually considered as being included in the assumptions for saturation. The final resultant can be found as described for the river wall, and should fall within limits as therein referred to.

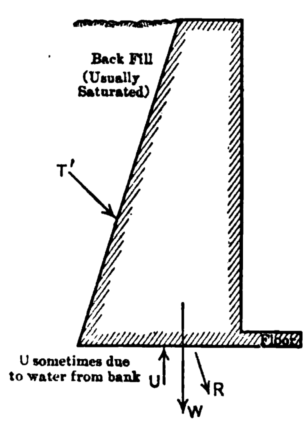


FIG. 152.—Conditions for Land Wall during Construction.

The extreme conditions (see Fig. 151) occur when the lock is pumped out for repairs with the lower pool at or above its ordinary stage. The forces are similar to those of the preceding case except that P is omitted.

A third condition (Fig. 152) occurs during construction just after the wall and floor have been finished and the backfilling put in, and with the cofferdam unwatered. This condition should also be investigated, but it is usually not as severe as the one described in the preceding paragraph, provided wet backfilling is not placed too high. This, however, is very liable to occur, and if the wall is not strong enough to meet this contingency special care must be exercised to see that the saturated material is kept within proper limits (see "Backfilling," p. 383). Sections of existing walls are shown on Pl. 45*a*.

Head- and Tail-bay Walls.—The functions of the head- and tail-bay walls are, first to support the concentrated pressure from the gates, and secondly, to

provide width for maneuvering the operating machinery, such as the capstans for opening and closing the gates. The latter condition usually requires a width of not less than 9 nor more than 12 feet behind the gate recess. In providing against the thrust from the gates it is assumed that the masonry for a certain length, usually taken as 15 feet, above the hollow quoins, acts as a monolith with an approximately similar length below them, making a total mass of about 30 feet. Thus, let Fig. 153 represent the head and tail walls on the river side in plan, the masses $ABCD$ and $FGHJ$ being taken as those which resist the pressure from the gates. The forces acting on $ABCD$ under normal conditions are the weight of the wall W, the pressure P of the upper pool on AE (usually not taken into account, as it is offset by a counterpressure on the outside of the wall, unless the dam is upstream of B), the resultant thrust T from the gate, and the underpressure U (see p. 363). T is equivalent to $\dfrac{Q}{2 \sin \alpha}$, where Q is the total pressure on the gate and its corresponding length of miter wall (the miter wall being usually designed to transmit its thrust into the main wall), and α is the angle between the sill and a normal to the wall as shown. T acts at 2α to this normal. At E we can resolve T into two components, S at right angles and X parallel to the axis of the lock. Their values are $S = T \cos 2\alpha$ and $X = T \sin 2\alpha$. These forces are usually investigated separately for their overturning effects, under the principles described or referred to for the river wall of the chamber in the preceding paragraphs. In addition to the foregoing, if the dam is downstream of $ABCD$, there will be a pressure from the upper pool on the outside downstream of E which will partly offset S, but which will cause a twisting tendency, as does also the component X. The effect of this twisting, however, can be and usually is neglected, except under very special conditions. The center of gravity of the mass $ABCD$ is assumed to be opposite to E, unless the design is such as to make this supposition too greatly in error. If the inside pressure P has to be taken into account, it is combined with T, as described below for the lower thrust wall.

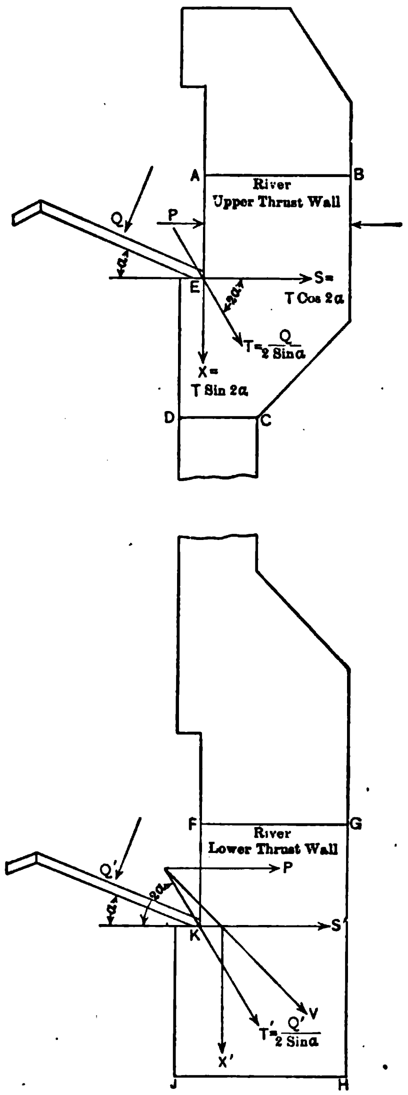


FIG. 153.

For extreme conditions the forces have to be modified by the effect of drawing off the lower pool, or by the effect of certain flood levels as described for the river wall of the chamber (p. 396). The foundation should also be examined for conditions of construction, when the dead load of the wall has to be supported, and if the lock gate is very heavy it may be necessary also to examine its overturning effect and the consequent additional pressure on the base at E.

For the lower river thrust wall $FGHJ$ (Fig. 153) we obtain forces similar in character to those at the upper end, except that the water pressure P on FK has usually no counterpressure on GH to offset it, unless the dam is downstream of G. Combining P with the thrust with T' from the gate, the resultant V can be found, and the transverse and longitudinal overturning components S' and X' become functions of V. They act where a normal to the lock wall through the hollow quoin cuts the line of V, as shown in the figure. The base JH is frequently prolonged downstream, making an inclined end so as to care for X' economically.

The strains on the thrust walls of the land side are generally similar to the foregoing, modified by the earth pressures.

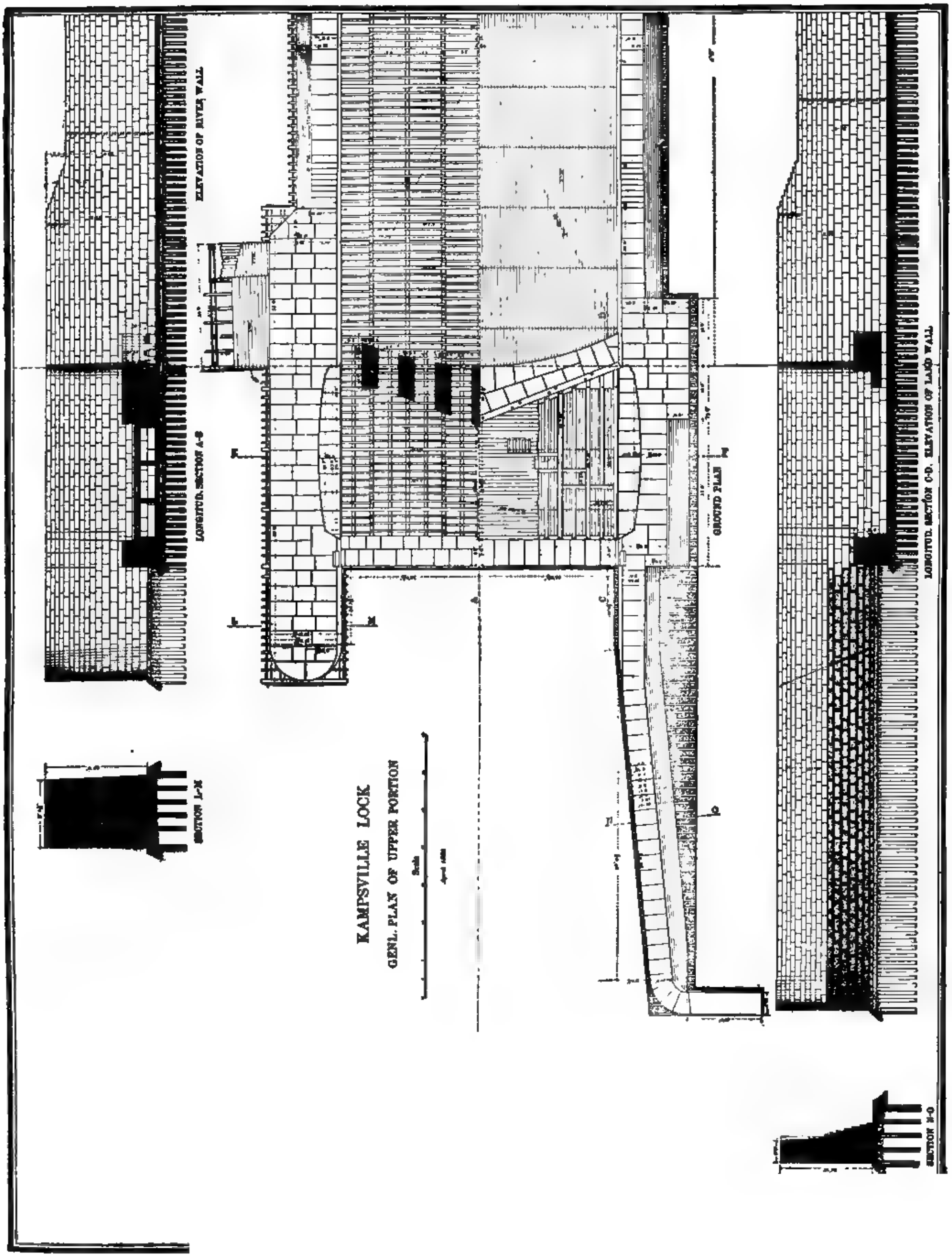
If the walls are of concrete, which is usually built up in sections with vertical joints from bottom to top, the sections should be arranged so as to avoid a joint near the hollow quoin, and to place behind the gate a monolith at least equal in mass to that required by the calculations. The foundations of the lower end of the walls must be designed with ample strength to resist force X', as, if any weakness exists the gates will tend to separate the concrete blocks at the nearest joint. This will sometimes occur in masonry locks also, several such cases being on record. One of them, which occurred within recent years on the Green River in Kentucky, is worth noting because of the unusual circumstances. The lock, which was built about 1840, carried a maximum head of some 15 feet. The lower end of the river wall, where the parting occurred, was built on squared timbers placed on bed-rock, a method of construction which appeared to be in vogue at that period, and of which several examples were to be found on the same river. The mass of the masonry appeared to be ample, but the wall had parted in an irregular seam across the gate recess and moved visibly as the water in the chamber rose or fell. The cause lay in the timber footing, which had been worn away where exposed to the current from the dam, thus reducing the base width. The wall was satisfactorily repaired by building a concrete buttress on the outside.

Upper Coffer Wall.—The upper coffer wall is needed only when repairs are to be made. Its top is often provided with a wooden sill which supports the needles used for the cofferdam, their heads being supported against a beam just above the upper pool; in later practice, however, the sill has been omitted, and the needles rest directly against the masonry.

THE LOCK

Section AB and Elevation of Land Wall

GENERAL PLAN, ETC., OF LOCK NO. 7, KANAWHA RIVER, W. VA.
(Built on rock)



GENERAL PLAN, ETC., OF THE UPPER END OF THE KAMPSVILLE LOCK, ILLINOIS RIVER, ILL.
(Built on piles with grillage.)

Let $ABDE$ (Fig. 154) be a section of a wall supporting needles CB, h the head of water on the needles, (usually assumed with the pool 2 or 3 feet above its low-water level), and d the height of the wall. The pressure on CB per foot run, supposing the needles to be vertical, is $P=\frac{h^2}{2}\times 62\frac{1}{2}$ lbs., of which two-thirds go to B. The pressure on BD per foot run, acting at the center of gravity of the quadrilateral of pressure on BD, is

$$Q=d\left(h+\frac{d}{2}\right)\times 62\tfrac{1}{2}\text{ lbs.}$$

Combining Q and the proportion of P at B into their resultant S, and finding the weight of the wall W, less the effect of underpressure (see p. 363), the position of the final resultant R may be determined as described for the river chamber wall.

The space behind AE usually remains filled with water when the head bay is pumped out, and this pressure tends to offset that on the upstream side.

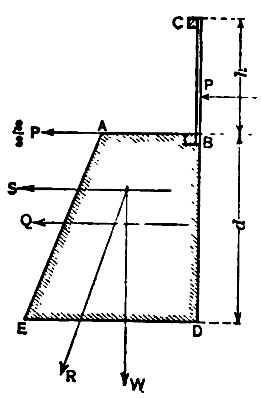


FIG. 154.

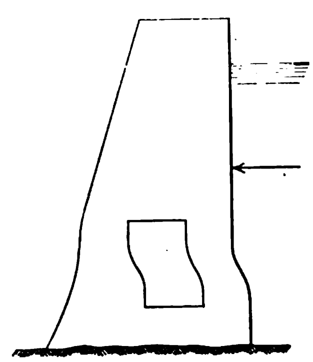


FIG. 155.

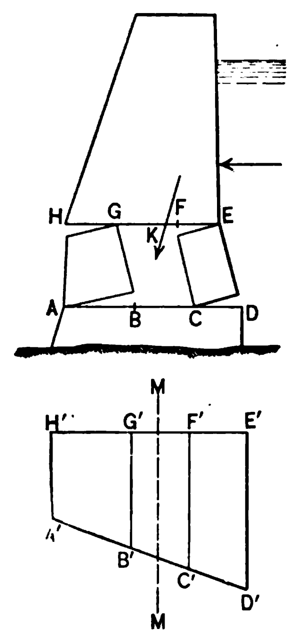


FIG. 156.

However, as it may be necessary at some time to empty the space, the pressure on AE should not be relied on as acting at all times.

Effect of Culverts.—Where culverts are used in the walls, they induce secondary stresses which, under severe conditions, will require special provision. Thus, if Fig. 155 represents a section of a lock wall, the thin walls of the culvert, if capable of taking tension, will act partly as restrained beams and partly as columns in resisting the horizontal components of the outside forces and the vertical loads respectively. The stresses can be investigated in connection with the theory of the elastic arch, or by considering the columns as restrained beams, under the assumption that the masonry

will resist tension. In construction, however, a joint is usually left along *AD* (Fig. 156); with stone, for convenience of the courses, and with concrete, in order to set the culvert form or mould. A joint is often placed also at *EH*, so that the capacity for tension becomes very uncertain, and the actual tendency will thus be for the walls to act as loose columns and to tip as shown in the figure. An analysis under this assumption takes the severest possible conditions, but is simpler and more conservative than the methods before mentioned. The horizontal shear along *EH* of the forces above that line is divided under this supposition between *EF* and *GH* in proportino to their areas, and the horizontal water or earth pressure on the back column *ABGH* (this pressure is balanced or else does not occur on the front one) is divided in proper proportion between *HG* and *AB*, the portion along *AB* going directly into the base, and thus not affecting further the stability of the column. The vertical loads from above *EH* act at *K*, where the general resultant of all the forces above *EH* cuts this line. Let *H'E'D'A'* (Fig. 156) represent this vertical load diagram, laid out from this resultant point. Assume that the portion *F'E'D'C'* acts on its supporting column through its own center of gravity, and that the part *MF'C'M*, obtained by dividing the load *G'F'C'B'* over the culvert by a central line *MM*, is uniformly distributed over *FE*. Add to this the weight of the column itself, and then solve for the final resultant of all its horizontal and vertical forces; this should pass within limits that will give a safe pressure on the toe (see p. 368). The forces on the other column *ABGH* can be similarly treated.

The foregoing analysis is open to mathematical criticism in certain points, and its assumptions for resistance are conservative, but it is believed wise not to place too much dependence on possible help from tension in the masonry owing to the uncertainties of construction. In some cases it will be found that the resultant for one column will be too safe, while that for the other will be unsafe. Under such conditions reliance must be placed on the fact that the wall will act as a whole and that overstrain on one part will be transferred to the other part, so that if the average of the resultants gives satisfactory results, the wall should be safe. Fillets or bevels at the corners of the culvert will assist conditions materially. In the case of high culverts the opposing water pressures on the top and the bottom of the culvert may affect the stresses considerably. The portion under the culvert between *B* and *C* must be of sufficient strength and depth to redistribute the loads from the columns over the foundation; this may require reinforcing rods or an additional thickness of masonry.

Walls in Rock.—Cases are occasionally met with where a lock is to be set into rock, as shown in Fig. 157 (see Pl. 45*a* for other examples). Some engineers favor using a gravity wall for such conditions, as shown by the broken lines *FH* and *HD*, on the ground that water may get behind it and produce an overturning thrust, while others are of the opinion that if suitable precautions are taken for drainage, a facing like *BCDE* from 3 to 6 feet in thickness is ample for the portion in the rock, and

that only the upper portion *ABFG* need be of gravity section.* If this plan is adopted, the joint *EF* between the masonry and the rock must be long enough to get a good grip on the natural foundation, and to overcome all tendency to sliding forward from the horizontal pressures on the rear. Probably the most notable example of this class of faced construction is to be found in the wheel-pits of the Niagara power plants. These were excavated in a more or less solid rock, which was water-bearing in some portions, as the supply canals and the Niagara River were close against or near the work. The rock was cut out by channeling and then faced with hard brick, lightly anchored to the rock (Fig. 158). The pits are over 600 feet long, and go down more than 175 feet below the surface water level. At horizontal intervals of 10 to 20 feet vertical drainage recesses a few inches deep and 2 to 3 feet in width were placed running from bottom to top of the facing, with occasional horizontal connecting drains.

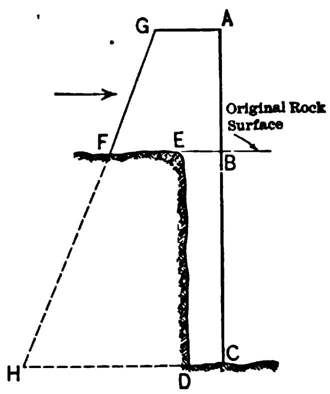


FIG. 157.

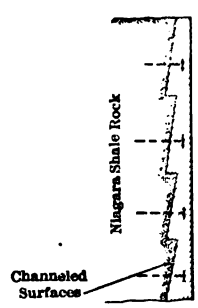


FIG. 158.

The water seeping out of the rock collects in these drains and pours into the tail races, the discharge being copious at some outlets, while at others, often close by, it is practically nothing. To have provided against the pressure of this water with a gravity wall would have required a section of enormous size; as designed, the brick facing is only about 24 inches thick at the bottom and 12 inches at the top, with a total height of 150 feet, and so well has the design taken care of the drainage that the inside faces of the walls are practically dry. The first pit was built prior to 1894, and has since fulfilled its purpose perfectly, so that its type of construction has had a long and successful test.† Examples of such facing applied to locks are to be found at several of the river locks of the New York State Barge Canal, St. Nazaire Harbor, and occasionally elsewhere, sections of some of the walls being shown in Figs. 159 to 161. If water-bearing seams are met with at special points, a system of drainage must be placed so as to serve all points of danger, and the water should be carried into the out-

* On the old Delaware and Hudson Canal (now largely abandoned) were some locks whose chambers were cut out of the solid rock, no facing being used for the walls except where necessary for the lock gates.

† Transactions Am. Soc. C. E., 1908, "The Niagara Power Plants."

lets of the main culverts, or to some place below the lower gates where it will not be choked by silt or débris, nor be affected by the change of water levels in the chamber.

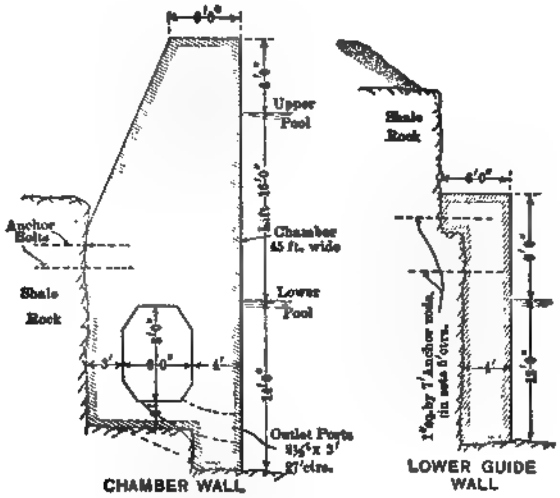


FIG. 159.—Sections of Walls of Lock at Stillwater, Hudson River, N. Y. (1911.)

Experience with lock pits excavated into solid rock has shown, however, that water-bearing seams are rarely met with, and that the formation is usually impervious to water from outside. Under such conditions a masonry facing such as that shown in Fig. 157, well anchored to the rock, will answer all ordinary purposes, especially if provision for drainage is made. Where such a facing is used, it is best to have the rock cut out by channeling or by special drilling, as this leaves a smooth surface for the concrete (see p. 584). If blasted out carelessly the seams and cracks will extend some distance back from the desired face,

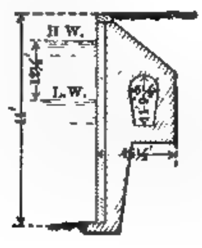


FIG. 160.—Section of North Wall of Lock at Little Falls, N. Y. (Mohawk River, 1911.)

FIG. 161.—Section of Lock Wall, Harbor of St. Nazaire, France. (1906.)

and the cost of removing all shattered pieces and of replacing them with concrete is often more than would have been the cost of channeling or extra drilling.

Battered Chamber Walls.—A method of construction sometimes adopted consists of building the chamber walls with a batter on the inner face of $\frac{3}{8}$ or $\frac{1}{2}$ inch to the foot. (Fig. 162 and Pl. 45*a*.) Few examples of this are to be found in America, where the walls are built almost invariably with vertical faces, but in Europe the type is common. The chief advantage lies in securing greater stability with the same cubic contents, and in the reduced liability to disfigurement. On the Kanawha River examples exist of each kind, and on those with vertical faces the walls have become seamed and scarred with the rubbings of craft, while those with battered faces showed hardly a scratch after twenty years of use. The reason is that with the latter type the bottom of a barge is usually the first part to strike the masonry, and whereas the top of the barge is protected with iron which tends to cut and scar the walls, the bottom has only the edges of the wooden planking with which to strike. The additional expense of construction for battered walls is small. It would be preferable, however, where these are used, to continue the batter from end to end of the lock for the sake of

FIG. 162.—Section of Lock Chamber with Battered Masonry Walls and Pile Fenders. (At Teglingen on the Dortmund-Ems Canal. Built about 1896.)

appearance, instead of making the head and tail walls plumb, as is the usual practice, since, with this composite type, the vertical portions of the walls become scarred by craft entering or leaving, and the corners of the walls and quoins become chipped off. The hollow quoins and gate recesses could be made plumb (being out of the way of boats), and the remainder of the walls could be battered.

The width of a chamber with battered walls is of course measured at the bottom so as to afford at all points the full dimensions required.

Slope, Reinforced Concrete, and Miscellaneous Types of Walls.—(See sections of walls on Pl. 45*a*.) In many of the old locks in Europe, as on the Upper Seine and the Yonne, and in a few of recent date, as on the Dortmund-Ems Canal, the chamber walls were formed by sloping the earth and covering it with paving or with a cemented wall. The floor was the natural foundation, paved where needed (Fig. 163). The head and tail walls were of masonry and of the usual vertical type. This method effects a considerable saving in first cost, but has not been found wholly satisfactory. More water is consumed and a longer time required for filling and emptying, and the boats occasionally get stranded on the slopes, where the latter are not properly protected

damaging themselves and the paving. The last objection has been remedied by using guide piles (Fig. 163), but these are also liable to be damaged. As this type of lock chamber does not permit filling through side culverts, but requires filling from one end only with consequent disturbance to boats in the lock, its use has been limited to lifts of about 8½ feet. For greater lifts a combination type has been satisfactorily tried within the last few years, consisting of a paved earth slope on one side and a masonry wall on the other, the wall being provided with a longitudinal culvert through which the chamber can be filled with a minimum of disturbance.

Many of the first locks in the United States were built of cribs of logs filled with gravel or riprap, with the faces planked where necessary. A cut-off wall of plank or sheet-piling was used, running from the heels of the gates into the bank to prevent water getting past. This construction is still to be seen in some of the Russian locks, and gives a cheap and effective, though temporary, service.

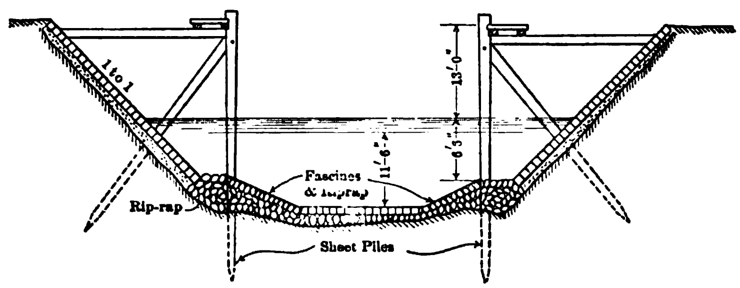


FIG. 163.—Section of a Lock Chamber with Paved Earth Slopes and Pile Fenders. (At Bollingerfähr, on the Dortmund-Ems Canal.)

Another method, of which examples were to be seen on the Fox River in Wisconsin as late as 1906, consists of head and tail walls of cemented masonry and chamber walls of dry rubble, faced with two thicknesses of jointed 2-inch plank spiked to posts set against the rubble. The woodwork usually needed replacing after about ten years, but the type was found very serviceable.

Still another type, which has been often proposed but rarely used as yet, is the wall of reinforced concrete, either with earth-filled pens, or of the plain L-type. The objections to it are practical rather than theoretical, and consist in the cost of construction, and in the serious effects of careless placing of the concrete or the reinforcement. This last-named objection is unfortunately often in evidence. Regarding the question of cost, comparative bids were obtained in 1905 for a lock on the Trinity River, Texas, one design being for the usual solid walls and the other for reinforced concrete walls. While the latter required much less masonry, the price bid per cubic yard was considerably higher, and the general summary proved that a contract for ordinary walls would be the cheaper. Similarly, on the New York State Barge Canal in 1907, contracts were made for the inside of parts of some of the lock-

walls to be composed of grouted rubble, the contract price for it being less than one-half of that for the surrounding concrete. Later, however, it was found that the various contractors when bidding had increased their prices for the concrete so as to cover the complications of construction due to the rubble filling, and the net result showed that solid walls would therefore have been equally as cheap. Before construction, one contractor after another applied for permission to supplant the grouted filling with solid concrete without increase of cost to the State, and this was accordingly done.

An example of reinforced concrete walls is to be found on the Merwede Canal in Holland, where a lock of this type was constructed about 1904–1906. The maximum lift does not exceed 7 feet and the chamber is 116 feet wide and made for an 8-foot draft. The use of reinforced concrete was estimated to have effected a saving of about 15 per cent. Another lock of this character, with a lift of 10½ feet and an available length of 900 feet, was recently completed on the Toura-Tobol Waterway in Russia, and was estimated to have effected a saving of 20 per cent.* (See Pl. 45*a*.) Several examples have been completed recently in Germany, as at Gleesen, Minden, and elsewhere, and guide walls of this type have been built on the Muskingum River in Ohio.

The comparative lightness of the reinforced concrete wall has made its use advantageous in several cases where the natural foundations were not suited for heavy loads, while for ordinary conditions it is possible that a design combining certain features of the gravity wall and of the hollow reinforced wall, similar to that of the Rhine-Herne Canal lock shown on Pl. 45*a*, would prove both economical and satisfactory. Such a design would avoid much of the expense for forms and labor which is the chief item of cost with the reinforced wall and yet would provide a sufficient mass to guard against the results of any carelessness in construction.

Floor.—The floor of the chamber, where artificial, is a most important part of the construction. Any weakness which it may develop can be repaired only by pumping out the pit and stopping navigation entirely, and in extreme cases it may be necessary to build a cofferdam around the entire lock in order to cut off the outside water. Such repairing, moreover, is always tedious and expensive, and it is difficult to make it thoroughly satisfactory. Too much reliance should not be placed on the ability of sheet-piling to stop seepage under a floor or a lock. Where it can be driven under the best conditions, as was done at the Wachusetts Dam, Mass., where each pile was planed and sunk through soft material with water-jets, it may prove very effective; but as these conditions cannot be secured on river-work, the chief function of the piling appears to be to obstruct the carrying off or the flow of material, and usually the leakage of water is considerable, even with the best care. Thus, on the upper end of a cofferdam for a new lock, triple-lap sheet-piling was driven to rock through several

* Transactions Am. Soc. C. E., 1904, Engineering Congress at St. Louis; Proceedings International Congress of Navigation, 1912.

feet of sand and mud, the total depth from the water surface being about 13 feet. The piles, which required a minimum of driving, were of good lumber, rough but not warped, and were carefully put down. In spite of the favorable conditions, however, when the slope of the river caused a head of about 6 inches between the upper and lower sides of the piling, (before the pumping-out of the cofferdam), a copious leakage was visible through the joints, and this continued until the backing was completed. Steel sheet-piling, while more reliable than the wooden, is also liable to some leakage.

The floor is subject to the downward pressure of the water when the chamber is filled, and to upward pressure from the lower pool when the lock is pumped out, and also, according to both theory and experience, to a certain amount of pressure from the upper pool unless the foundation is of impervious rock (see p. 361 and after). This pressure, if counteracted by the weight of the floor alone, would require a large amount of masonry, as on the Bougival lock on the lower Seine, where a portion of the floor, with a chamber width of about 55 feet, is $14\frac{1}{2}$ feet thick. To avoid this expense many expedients have been used, such as employing piles to hold down the floor; using a flat or an inverted arch; leaving holes in the floor to let the restrained water escape into the chamber, or placing longitudinal drains under the floor so it can escape into the lower pool; and finally, with locks on gravel, using only a loose covering of paving.

Wooden and Concrete Floors.—The first and second expedients are the ones most commonly met with, either alone or in combination. If a wooden floor is used, stringers are bolted to the sides of the piles (preferably with hardwood pins because of their non-rusting) and a double layer of planking (each one usually 2 inches in thickness) is spiked on. In a few cases the stringers have been laid on the pile heads, and drift-bolts have been driven vertically to hold them down. The underpressure, however, has in several locks forced up the bolts, and caused a leaky floor. The lower layer of timber is usually calked, and this precaution is a wise one, as unless the top surface is made impervious by a layer of concrete or by other means the water will force its way through the smallest openings in the joints of the plank and will gradually enlarge them, so that if the natural foundation is of light material it may in time be undermined. Wooden floors, however, should never be used unless in exceptional circumstances, as they are non-permanent, both as regards metal fastenings and planking (see p. 385), and usually cost almost as much as concrete. If they have to be employed, the surface should be formed of a layer of concrete 3 to 6 inches in thickness, embedded around spikes driven into the timbers. This will prevent surface wear and secure a tight floor.

With a concrete-and-pile floor, the pile heads are usually notched and embedded 12 to 18 inches in the concrete, and the piles are spaced closely enough to afford support to the concrete between them, the concrete thus carrying its load as a beam, and the piles, as with wooden floors, taking care of the upthrust. This spacing is usually made 3 to 4 feet centers, and the concrete runs from 18 inches to 3 feet or more in thickness,

depending on local conditions, as well as on the experience or the confidence of the designer. Where piles carry tension, they should always have the bark removed, as if this is not done, the bark will gradually soften under immersion and will form a slip-joint around the piles.

Inverted-arch Floors.—Much additional strength is gained by placing the concrete as an inverted arch. On the river locks of the New York State Barge Canal (1905 and later) as well as on the locks of the canal portions, nearly all floors on porous foundations were formed of inverted-arch concrete laid around notched piles. The locks were 45 feet in chamber width, and the floors were in most cases formed of 3 feet of concrete (the lifts ranging from 8 to 15 feet), placed around piles each carrying from 12 to 16 square feet of area. The floor of the Yosts lock on the Mohawk River, laid on gravel and with no piles, was made 5 feet in thickness. The rise of the arches was generally 3 feet. The floors of the head and tail bays were in some of the cases made flat, the piles taking care of the underpressure. These arrangements were successfully tested at several points where the lock chambers had to be unwatered during construction, and were subjected at times to a head of 20 feet. (See p. 438.)

The inverted-arch floor without piles has been largely used in Europe, sometimes with cut stone and sometimes with concrete, and examples are also to be found in America, as on the Mohawk River, and elsewhere. With the former material flat arches have been constructed in some cases and curved ones in others. Thus at the locks of Dinant and Anseremme, on the Belgian Meuse, with chambers 39 feet wide, the floors were formed of inverted arches of 2 feet rise, the voussoirs being 15 inches deep and laid on a bed of masonry of a least thickness of $1\frac{3}{4}$ feet. Below this was a bed of concrete 2 feet thick. At the Suresnes lock on the Seine, finished about 1885 and built on clay with a lift of nearly 11 feet, the rise was made 20 inches in 59 feet, the arch proper being laid on a solid bed of masonry. A rise of 1 in 20 is customary in French practice. DeMas recommends for a lock 17 feet wide, on a porous foundation, a floor thickness of $2\frac{1}{2}$ to 3 feet, and if on a very porous foundation, 4 to 5 feet.

The approximate total thrust T of the arch in a segment of floor 1 foot in width (measured along the lock) and of the full depth of the masonry, can be found from the formula $T=PR$, where P = the pressure of the water per square foot on the under side of the floor and R = the average radius in feet. P is usually taken as the average pressure, and from it can be deducted the weight of the masonry. The floor should be made strong enough to carry the head when the chamber is unwatered, and the walls must provide suitable abutments for the thrust and be secure against its horizontal pressures. The compression in the floor arch may be taken at 300 pounds per square inch for normal conditions, with an increase up to 25 per cent for extreme conditions.

Reinforced-concrete Floors.—In order to reduce the thickness of the concrete, reinforcement has occasionally been employed, as shown on some of the cuts of Pl. 45*a*. In such cases piles are generally used, and serve to hold down the masonry as

described for plain concrete floors. At the Bremen lock, however (see p. 413), no piles were used.

Open Floors.—Where the natural foundations have been of gravel, the material in some cases has been left uncovered, so as to form a porous floor, loose paving being used where necessary to keep the water from eroding the surface. Examples are to be found on the Ohio River, on the Elbe in Bohemia, and elsewhere. This method allows more or less escape of water under the foundations when the chamber is full, but as it has been employed so far with heads not exceeding about 10 feet, no harm has resulted, and the type has proved satisfactory. In one special instance in America, on the Allegheny River, the main floor was placed 10 or 12 feet below the lower miter-sill level, and surrounded on all sides with concrete, exactly like a box without a lid. This box was then filled with gravel or other heavy material, and covered with plank spiked to joists, the idea being that the weight on the floor would counteract the upward pressure.

Rock Floors.—Where the foundation is of rock, it is customary to use it for the floor surface, without any artificial covering. In rare cases, where the rock is very soft, it may be desirable to use aprons opposite the culvert outlets or to carry down the footings of the walls there, but it should be remembered that violent movements of water in the chamber are of very short duration, and only tend to cause erosion at a few points. Experience has shown that ordinary rock is amply able to serve all purposes of a floor, even under very high heads, and that an artificial covering for it is a needless expense. In one instance where a rock floor had been covered with a foot of concrete, seepage from the bank leaked through seams in the rock during construction and raised and cracked the concrete for nearly half the length of the lock. Where such seams are found they should be scraped out and be thoroughly filled with mortar, or the water may escape through them when the chamber is full and interfere with the operation. In one case where this precaution had been omitted winches had to be used to pull the upper gates open, owing to the copious leakage. (See Chapter X, "Undermining.")

Drains in and under Floors.—In several locks provision has been made for relieving the underpressure from the upper pool by leaving small holes in the floor, or by providing riprap drains connecting with the lower pool. Where the former method is used the holes are usually made from 1 to 2 inches in diameter, and at their lower ends are placed a few cubic feet of sand and gravel which act as a filter to prevent the washing away of the surrounding material (Fig. 164). Another method is to use a socket valve at the top with a loose ball which will let the water flow in the upward direction only. These valves and fittings should always be of brass or bronze; iron parts have been found to rust together quickly and prevent any movement of the ball. The holes may be placed from 5 to 10 feet apart. The great lock at Ymuiden on the Amsterdam Ship Canal (Pl. 45*a*), built on sand and with a width of 82 feet and a depth on sills of about 32 feet, has its masonry floor provided with gravel-filled holes

for relieving underpressure, and other examples are to be met with both in America and in Europe.

The second method above referred to consists in putting under the floor longitudinal drains of 1 or 2 square feet in area composed of pieces of small riprap and usually spaced from 4 to 12 feet apart, and leading from near the upper miter wall to some point of exit—frequently in the emptying culverts downstream of the valves—at the lower end. These drains should always be surrounded by a filtering-bed of fine gravel or coarse sand, or of both mixed, or the surrounding material if light will gradually seep in and fill up the spaces in the riprap. This at least has been the experience with many such drains laid in sandy railroad cuts. Vitrified tile pipe, laid in sand and gravel, has also been used instead of riprap, as on the Bremen lock, built about 1908, the floor of which was made of 18 inches of reinforced concrete, with a 3-ft. bed of sand and gravel (in which tile pipe were laid) underneath. Anchor rods with large washers were also set into the natural foundation to hold down the floor. The width of chamber was 40 feet.

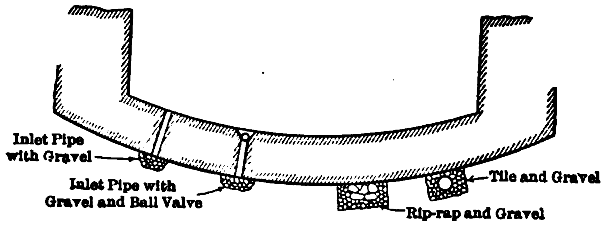


Fig. 164.—Methods of Under-draining Lock Floors.

The chief objection to these drains with sedimentary rivers lies in the fact that the outlets are very liable to become silted up, thus destroying their efficiency. Where emptying into the culverts the rush of water would assist in keeping them open, but during prolonged floods when the lock is not in use, the deposits in the culverts may reach several feet in depth, and would probably effectually choke the outlets.

There appears to be little or no actual evidence as to the value of these methods of underdraining floors, but under suitable conditions they seem to offer a reasonable promise of efficiency.

General Remarks on Floors.—The designing of a floor is usually unsatisfactory from a theoretical standpoint because of the unavoidable lack of knowledge of the actual pressures it must carry. The failure of a floor is a difficult and expensive matter to remedy, although few cases are known where it has led to any failure of the lock walls themselves, and broken floors have sometimes lasted through many years of use. Some examples of trouble with floors will be found in Chapter X. However, undue risk should never be taken in designing a floor; it should be made able to resist safely any pressures that may be liable to occur, especially as the difference in first cost

between a strong floor and a weak one is rarely excessive, and the unquestionable security of the former means an insurance against future damage and expense.

Sump Basin.—Where conditions permit, it is well to provide a " sump " or drainage basin in the floor, for the use of the suction pipe when the chamber has to be pumped out, as the water can then be drawn entirely off the floor, instead of remaining on it to the depth of a foot or more, as is usually the case. This sump should be placed preferably close to the lower end.

Protection from Erosion.—A space of 30 to 60 feet (depending on the lift) below the lock, when the foundation is not rock, should be protected with an apron crib or with plenty of riprap, in pieces containing ½ to 1 cubic yard, so that no washout will be caused by the discharge from the valves. The tendency to erosion at this point is always severe, and unless checked may endanger the foundations of the walls. Sheet-piling is excellent, but the crib or riprap is needed also. On certain locks on the Welland Canal in Canada, with 14 feet on the sill and of 14 feet lift, built on clay and emptying through valves in the gates, an examination after many years of operation showed that only the sheet-piling across the lower ends had saved the walls from being undermined, as deep scour had taken place. The cavities were filled later with riprap, but stones of nearly a cubic yard each had to be employed before safety was assured. Similarly at one of the Erie Canal locks with a lift of 10 feet and a depth on the sill of 6 feet, built on soft clay and emptying through valves in the gates, traces of erosion were found 80 feet below the lock. It was only necessary, however, to use paving for about 40 feet of that distance.

This part of the structure should be carefully watched for some time after the lock is put in operation, so as to locate and remedy any erosion that may develop. It is a good plan in fact to let the water discharge continuously for a week or two, as this will usually wash out any soft material very quickly, and the damage can then be promptly remedied once and for all.

Where the depth of water is considerable, as for instance a 10-ft. channel depth compared with one of 6 feet, the disturbance is reduced in proportion, the larger body of water tending to reduce the velocity more quickly. This is very noticeable at deep-draft locks.

Elevation and Angle of Miter Sills.—The lower miter sill in American practice is usually placed at the depth below pool assumed for the navigable channel; thus, if a channel depth of 6 feet is contemplated the top of the sill is placed 6 feet below pool. It should never be less, as in a dry season, with little water running, the entire pool will closely approximate the level of the crest of the dam below, and if the latter is leaky the depth on the sill will be accordingly reduced. For this reason the sill should be placed as low as practicable, since an excess of draft is much better than too little, and is of very great advantage in allowing deep-draft boats to enter or leave the chamber without " choking " the water. This ease of entry and exit is much valued on European

waterways. Thus on the lower Seine the depth on the lower miter sills is in some cases 3 feet more than the channel depth; on the Dortmund-Ems Canal, 1 foot 9 inches more; and on the River Weaver, 5 feet more. A Swedish Royal Commission on Navigation recommended that the depth on miter sills should exceed the draft of the largest boat by at least 10 per cent.*

This principle of excess depth is generally adopted for the upper miter sill where the dam is fixed. Thus if a 6-ft. channel depth is desired, the sill is frequently placed from 8 to 12 feet below the upper pool. Usually this can be done with no additional expense, for while it requires higher lock gates it saves masonry in the miter wall. Should the river have to be improved at any future date, so as to afford a greater navigable depth, a sill so placed will not have to be disturbed, and a large saving will have resulted with no greater first cost.

The sills of the upper and lower coffer walls are usually placed at the same elevations as the upper and lower miter sills respectively.

Where the lock is connected with a movable dam the upper and lower sills are generally placed at, or nearly at the same level, usually from 6 inches above to a foot or more below the sill of the pass, depending on the navigable depth required and local conditions. By this arrangement the lock gates can be left open when the dam is lowered for the winter, preventing the chamber from filling with deposit and at the same time affording more area of discharge. It also permits lockages to be made when the dam is being raised, and when boats therefore cannot pass through it, but must go over the sills of the lock before the upper pool has become filled.

The economical angle of the sills, or the angle which permits the minimum of material in the gates, will usually be found to lie between 19° and 21° with a normal to the face of the walls. For this reason an angle of 20° is frequently adopted.

Recesses for Lock Gates.—The recesses for the gates should be at least 18 inches longer than the gates when these are open, so that the water can flow easily behind them when closing begins, and should afford a clearance of not less than a foot between the gates and the wall, to allow space for floating sticks and similar débris, which would otherwise hinder them from opening fully. Insufficient clearance behind a gate entails a constant and needless labor on the part of the lock-tender of removing such débris. The clearance under the gate can be from 6 to 12 inches.

Miter Sills and Walls.—Miter sills are usually made of 10×10-inch or 12×12-inch timbers, with their tops a foot or 18 inches above the floor. In the older locks they are frequently 16 to 18 inches square and 2 feet high or more above the floor. Their object is to provide an elastic cushion for the bottom of the gates. They should be well bolted down, since they are sometimes subjected to a lifting pressure from the gates, and when once started the upward water-pressure is of course added. This has occurred at the locks of the St. Mary's Falls Canal, at those of the Louisville and

* Proceedings International Congress of Navigation, 1912.

Portland Canal, and elsewhere. The sills are also subject to blows from the gates in closing, and sometimes, where the dams are leaky and the water low, to the strain of boats being dragged over them. They should not be lap-jointed at the miter, nor should they project under the lock wall, but should be constructed so that one piece can be taken out and replaced without difficulty, and without disturbing the other.

Although timbers larger than 12×12-inch have been used, this size and even smaller ones have proved ample in practice. There appears in fact to be no valid reason against omitting wooden miter sills altogether, and allowing the gates to close against the face of the masonry instead. This has been done on some locks of recent construction, such as some of those on the Bohemian Elbe, where an elastic contact is obtained by strips of wood about 3 inches in width, bolted along the bottom girders of the steel lock gates. These strips close against miter sills of cut stone. This arrangement avoids the troubles incident to wooden sills, such as splintering, lifting, etc., as masonry will stand a considerably rougher usage than timber.

The lap of the gate on the sill is usually made about 6 inches, with another 6 inches for general clearance underneath, and these dimensions have proved ample for all classes of gates.

For bolting down wooden miter-sills $1\frac{1}{4}$-inch bolts are commonly used, set 4 feet or more into the masonry, and with plain washers at the lower ends. For large gates the length is usually made more than 4 feet.

Where the sills are subject to wear from too-heavily laden boats scraping over them, as on the Monongahela River, the tops have sometimes been protected with iron angles or channels. With wooden or stone sills the upstream edge of the sill should always be rounded slightly, to prevent splintering or chipping from this cause.

Where the miter walls or the coffer walls are shallow and consequently of small mass, they should be well secured to the foundations with bolts or otherwise, or the water may leak beneath and force them up. Several accidents of this nature have occurred in this country.

Culverts.—See paragraphs "Culverts and Valves" in the next chapter.

Hollow Quoins.—In locks of cut stone the hollow quoin is cut directly in the pieces, which are set on each other as the building progresses. Especial care must be taken to see that they are set perfectly plumb and level, or it may be necessary to trim the whole quoin after all the stones are in place. The gate bears directly against the surface of these stones, which are made concave to suit the radius of the heel, the center of rotation of the gate being placed a little upstream of the center of the heel, so as to allow the gate to swing clear of the quoin during movement. (See "Center of Rotation" in the next chapter.)

With concrete locks the quoins have sometimes been made of cast iron, 1 inch or $1\frac{1}{4}$ inches thick, in sections 5 or 6 feet long, planed for the gate-bearing surfaces, and bolted to each other and to the walls. (See Pl. 50.) The sections should be planed

on all edges and sides except at the back, as this not only secures a neat appearance when in place, but also facilitates the aligning during construction. Lugs and turned bolts, set either on the face or on the back of each section, should be provided to fasten and hold the pieces to each other during erection. It is, however, difficult to keep the pieces in exact line, and they cannot be trimmed off or adjusted later. They are also expensive, and for these reasons they have not been used in many concrete locks, the quoin being preferably formed directly in the masonry, which can be trimmed to line if needed. Probably the best way in either case would be to leave a suitable recess, set the gate, and concrete up the recess, using the gate heel as a form. In crib locks the quoin is usually made of a timber set on end, and in Europe granite is often used.

On some of the older masonry locks in America the lower portions of the stone quoins have worn away from 1 to 2 inches after some forty years or more of service, due presumably to the tendency of the wooden gates to sag at the toe and thus grind into masonry.* Cast quoins have been used to good advantage to replace such worn portions. It is doubtful, however, if this wear would take place with the stiff modern gates of steel.

Probably the best shape for hollow quoins for ordinary locks consists of an arc of the same radius as the heel of the gate and about two-thirds of the semicircle in length, as shown on Pl. 50. This shape gives a wide bearing for the gate thrust, and affords support when the gate is struck from above by a boat, and provides an excellent protection against débris. It is very necessary to design the quoin and the heel so that débris cannot get between and strain the gate. One of the large gates of the Poe lock of the Sault Ste. Marie Canal, Michigan, where there is a space of some inches between the quoin and the gate when the latter is open, was strained at one time by a loose piece of towline getting caught, and damage was only averted by the lock-tender noticing that the gate showed signs of distress as the lock began to be filled. The lack of this protection is the great objection to the so-called flat quoin, in which the face of the masonry is straight and not hollowed out, and the heel of the gate is shaped to bear directly against it. (Fig. 165.) This type has been used to a considerable extent in Europe and occasionally in America, but its advantages appear to be greater in theory than in practice, and do not equal those of the standard circular quoin.

Another design used in Europe with metal gates consists in thrust castings and bearing plates at the end of each beam, as in Fig. 165*a*. The intermediate spaces are left open, and the seal is obtained by a wooden strip as shown.

Where the gates tend to float up during floods, or where the up-pressure on them is large, the tops of the quoins should be beveled for 6 or 8 inches down, as several cases

* On locks 6, 7, and 8, Kentucky River, built of oolitic limestone, the quoins toward the bottom of the gates wore away from 3 to 6 inches, and have been filled out where necessary with concrete, the stones having been chiseled away to a depth of 12 to 16 inches in order to secure a larger mass. These locks have wooden gates.

have occurred where the flotation pressure from the gates has split off pieces of masonry there.

Wing Walls and Dry Drains.—The upper and lower ends of the land wall are usually provided with wing walls, running into the bank from 30 to 60 feet from the face line, with the object of preventing the water from cutting behind the walls. Where the upper wing wall does not rest on rock, sheet-piling should always be driven along the upstream side, but with the lower wall this is not necessary. They frequently have to act as retaining walls also, and should be designed accordingly. Sometimes they are joined to the bank by a timber crib, in which case the latter should be sheathed inside and filled with tamped earth or clay. Riprap should never be used to fill it, on account of the danger of leakage. Where no dry drain is placed behind the chamber wall the upper wing wall can be made shorter, and in some cases it has

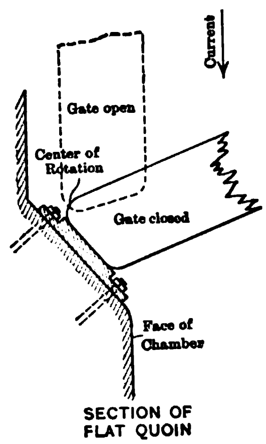


FIG. 165.

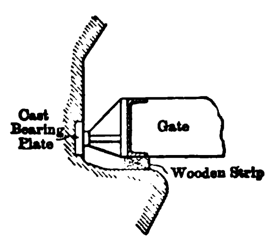


FIG. 165a.—Hollow Quoin of the Libschitz Lock, River Moldau.

been omitted altogether. Where upper and lower guide walls of solid construction are used, as for instance with guide walls of masonry, it is often allowable to omit wing walls, since the danger of injurious leakage behind the lock wall becomes less in proportion to the increase in distance through which the water has to travel.

Dry drains are used for the same purpose as with retaining walls, namely, to carry off any water that may collect and cause undue pressure behind the wall. Usually they are made of riprap, from 2 to 3 feet in width, and extending from the bottom of the wall to near the top, and from the lower end, where they connect with the river, to near the upper end. The back of the wall is thus open to the pool below, and the only obstacle to prevent the upper pool from flowing round is the upper wing wall and its backfilling. We have met with one example where the upper pool in high water forced its way around, passing under the paving to the drain, but fortunately without causing serious damage. If the drain be omitted, the water must force its way through a much greater mass of earth, and the gain in safety would appear to be worth more

than the doubtful utility of putting in a drain which the river will sooner or later choke with sediment. It may be added that the land wall in any case is usually designed to support the earth under the conditions of greatest pressure, that is, without any relieving drains, although these may be of service during construction. For these reasons many engineers do not favor their use. If they are put in, they should always discharge directly into the lower pool and not into the tail bay through a pipe or culvert, since, if the latter arrangement is adopted, it will not be possible to pump out the lock.

Backfilling. (See p. 383.)

Paving.—The space behind the land wall and between the wing walls is usually paved, to prevent floods from washing out the backing. This paving should preferably be laid on a bed of gravel a few inches in thickness, and may be composed of small blocks, 10 or 12 inches deep, set on edge, or of large stones laid flat, or of concrete. If the first style is used, it is a good plan to grout the joints with cement, or there will be a constant growth of grass and weeds in them. Where large stones or blocks are used they are difficult to reset if any sinking occurs, and for this reason are not favored by some engineers. With a new lock the paving should never be laid till the filling has been in place for one or two years unless the supporting material has been thoroughly compacted, or it may become badly disfigured by settlement. Temporary protection can be secured by the use of riprap.

With rivers subject to floods so high that when the lock walls are submerged there is still a head on the dam, particular attention must be paid to protecting the backfilling at the lower end. At this point there is usually a change of level in the grading, and the water in floods consequently pours over the slope and will quickly erode it unless it is thoroughly protected. Instances will be found in Chapter X where a river has made for itself a new channel behind the masonry from such a cause, and has left the dam useless in midstream. In one or two cases, as at Lock No. 5 on the Little Kanawha River in West Virginia, an earth spur-dike protected with riprap has been built out opposite to or a little below the crest of the dam and on the land side of the lock, with its top near or above flood level and running from the back of the masonry to high ground. This dike reduces the flow over the backfilling somewhat and thus assists in protecting the grade downstream. The nose under such conditions must be heavily paved, as the water creates a violent current around it, and its upstream and downstream sides must also be protected.

Temporary Cofferdams.—Arrangements at each end for closing the lock for repairs should always be provided. A method long in use for this purpose consists in placing timber beams horizontally, one on the other, across the head and tail bays. The ends of these timbers rest in vertical recesses cut in the walls, and they are supported at intermediate points against posts connecting with and braced to the masonry. This method is in use on certain canals in Canada with locks 45 feet in width and carrying 14 feet or more of water on the sills; on the Warrior River in Alabama with locks 52 feet in width

and with $6\frac{1}{2}$ to 7 feet on the sills; and at other localities. The placing of such a cofferdam, however, is slow, and requires large timbers. A needle dam of the Poirée type is also used for the same purpose, and is preferable in many ways, as it is simpler and more easily handled, besides requiring less timber. Where this type is used a steel or timber beam is generally employed for the top support, running from wall to wall, with its ends resting in horizontal slots. The needles or vertical planks rest against this beam and against a wooden or masonry sill on the upper coffer wall.* The slots should be placed somewhat above pool level, so that the beam can be put in at any ordinary stage of water. For very small locks a simple 12×12-inch timber will suffice; for locks of 36 feet in width the timber must be trussed or provided with a middle strut and a tie-rod, while for locks of wider opening the timber is usually cut into several pieces, or steel beams are used resting on movable iron trestles hinged to the coffer wall. In this case counterforts or buttresses may have to be built out from that wall to support the downstream ends of the trestles. This type of cofferdam can usually be put in place in a few hours. Portable timber A frames have been used very successfully instead of trestles.

Another, and in many ways a better method, is to omit the trestles, which sometimes rust away before they are needed, and to span the opening with a horizontal truss supported on temporary posts, and designed so that it can be used in two or more sections for easy handling. Trestles have also been found to possess the disadvantage of liability to injury when the lock entrance has to be dredged.

On the locks of the New York State Barge Canal (1905 and later) with chamber widths of 45 feet and not less than 12 feet on the sills, heavy steel beams or girders swinging horizontally were provided for the upper supports for the needles. This was done chiefly with the idea of using the beams as protectors to the gates by closing them each time the gates were to be closed. The method, however, is expensive and involves the use of considerably more metal and masonry than the tried and customary types.

Where no current can be created to hold the needles down when being placed, it will be necessary to use some means of weighting them. They can often be held down by putting sacks of earth or gravel against their feet.

Guide and Guard Walls.—Both the upper and lower entrances to a lock should be provided with a guide wall, and the upper entrance, where fixed dams exist, should have a guard wall also. A "guide wall" is the wall built in prolongation of the land wall of a lock, while a "guard wall" is placed on the river side of the lock entrance. The purpose of the guide wall is to assist boats in entering or leaving the lock, particularly while waiting for the lock to be made ready, and these walls are provided with ladders

* The needles are usually formed of planks or timbers of rectangluar cross-section, but needles of tongued-and-grooved lumber have occasionally been used and have proved very satisfactory, especially where the tongue and groove were made of hardwood strips, spiked to the needle. These strips are not as liable to become scarred or to break during handling as are tongues and grooves cut in the solid timber.

and line hooks and snubbing posts so as to permit deckhands to climb up and moor the boats. The object of guard walls is to prevent the current from carrying entering craft out into the river above the dam, and, when used at the lower entrance (which, however, is not a general practice), they protect the lock entrance from the violent currents caused at certain stages by the overpour of the dam. In constructing a lock it is usually best, unless the lift is great, to omit the lower guard wall, as cases are rare in which one is required. If found necessary it can be constructed later, but the probability of its being needed is usually remote.

A guide wall should be flush with the chamber face where it meets the lock walls and may either continue in the same line or flare landward from it. The former alignment is much preferable, however, especially where the lock is to be used by towboats and barges, as the fleet can then be pushed straight into the chamber. The length of the wall should be not less than the length of a tow which can pass through at one lockage, and the coping of the upper wall should be made level with the coping of the lock wall, so that boats can make use of it during floods up to the last moment. The coping of the lower one can be placed somewhat lower than the lock walls, but should be a few feet higher than the highest navigable water, so that boats can rest against it without their projecting guards extending over its top.

Guard walls may be either continuous or in separate sections with spaces of from 8 to 16 feet between, their tops being connected by light walkways or bridges. A space of from 10 to 15 feet should be left between the upper end of river wall and the guard wall so that drift, which is always collecting in the upper lock entrance, may be disposed of. (See pp. 423 and 424.)

In Europe guide walls are rarely used, and often only when they can be utilized as docks also. One reason appears to be that the locks are placed wherever practicable in derivations or side-cuts, and therefore are sheltered from currents, and another reason is that very few of the rivers approach in velocity or range of floods the Western and Southern rivers of the United States, upon which most of our locks and dams are found. On the other hand, on the Kanawha River in West Virginia—a typical Western stream and with a large traffic—commerce was carried on for many years with practically no guide walls, although the inconvenience to craft at times was considerable.

Types of Guide Walls.—Formerly timber cribs filled with stone were used almost exclusively for guide walls in America, but in recent years practically all new walls have been made of concrete, and in most cases as fast as the old timber structures have decayed they have been replaced by concrete walls, either built on parts of the old crib below low water which had not deteriorated to any marked extent, or built directly on the foundation originally occupied by the crib. As the latter method entails the use of a cofferdam the tendency has been quite often to raze the old crib to a point a foot or two below pool level and begin the new work on the remainder of the old foundation. In some cases, however, there is a serious danger of the latter

compressing under the added weight, and the timbers if exposed to a current may sooner of later become worn away to an extent which will endanger the superstructure. In some localities stone masonry has been used, but as a rule concrete has been used for both guide and guard walls.

Guide walls are usually built with a section similar to that for a retaining wall. If there is a back-fill or terre-plein behind the wall the coping width may be reduced to 3 or 4 feet, but for convenience and safety in handling lines this width should not be less than $5\frac{1}{2}$ feet where the back-fill is absent. Where the foundations are not on rock and are exposed to a current from the dam the use of piles is desirable in most cases, owing to the tendency to undermining. The walls may have vertical faces or they may be slightly battered; many engineers prefer the latter type, as the bilges or bottom corners of barges then rub along the wall and the iron-bound corners at the tops of the gunwales are in consequence less likely to scar and abrade the masonry. Drainage holes through the walls are sometimes provided if the foundation is rock.

On the Muskingum River in Ohio reinforced concrete guide walls and guide cribs have been built and are said to have proved satisfactory.

At several of the locks of the New York State Barge Canal, both on the river and the canal sections, concrete guide walls 300 feet long were used, resting, where the foundation was rock, in part or wholly on isolated piers from 2 to 3 feet in thickness and about 10 feet centers, only the upper portions of the walls being made continuous. In most cases the piers and the upper portions were well bolted together, and the piers were also bolted to the bedrock, to avoid displacement from blows from the heavy barges. Where the rock was sufficiently high it was merely faced with concrete from 2 to 3 feet in thickness, anchor-bolted, the construction being generally similar to that shown by Figs. 157 and 159, pp. 405 and 406. At Lock No. 21 on the Cumberland River is an upper river guard wall, 160 feet in length, of similar design, the supporting piers being 10 feet in length and extending to the full height of the wall, with a space of 12 feet between each pier. These spaces were closed with curtain walls of reinforced concrete 3 feet in thickness, extending from 1 foot below normal pool level to the tops of the piers.

The foundations for masonry guide and guard walls have sometimes been successfully built in water without a cofferdam. In France a box form has been used, sunk on the bottom, with layers of mortar and stone put in alternately by a diver. Concrete can also be lowered through a chute as described further on in this chapter under the heading "Laying Concrete under Water," while at certain of the dams in the delta of the Nile of comparatively recent construction rubble stone was thrown into submerged timber forms and the voids were then filled with grout.* With a rock foundation it is of course necessary to make sure that the surface is properly cleared of loose material before lowering the concrete or stone through the water.

* See Chapter IV, under the heading "Weirs on the Nile."

View of an Upper Lock-gate of Timber, with Triangular Guard Cribs.

Rear View of a Lower Land Guide Crib of Timber

View of a Lower Guide Crib Formed of Separate Timber Cribs with Grillages.

View of an Upper Guide Wall Formed of Timbers Spiked to Piles, Replacing an Old Crib.

Timber Guide Cribs.—As has just been stated, the use of wood in river structures has been largely abandoned, but in districts where timber is still plentiful it is occasionally used for guide and guard cribs, as well as for fixed dams. (See Pl. 46*a*.) Where wood is used a very suitable and convenient size for the timbers is 10 inches square, with sets of ties 10 feet apart, spiked together with a drift bolt at each intersection. Sometimes these timbers are dapped or notched into each other, but experience has shown that equally good results are obtained by simply spiking the ties and stringers upon each other, with butt joints where the latter come end to end. In putting in the stone, large pieces should be picked out and set with flat faces on edge against the openings between the timbers at the front and end of the crib, as this will give a much neater appearance than if the stones are left as they are dumped, and the extra cost is very small. The tops of the cribs are usually made 6 to 10 feet wide, and the width of base not less than one-half the height. As the vertical wedge of the filling always tends to push the crib out at the top, the face should be battered $\frac{1}{4}$ inch or $\frac{3}{8}$ inch to the foot; this may be done by setting each timber back as the building progresses, and the method has been found to be very effective in retarding the forward settlement. Diagonal bracing is also useful in this respect. Where high cribs are built with vertical faces, and no provision is made against undue weight upon the timbers, they will speedily begin to settle both forward and downward, and in a few years may lean over 12 inches or more at the top, and be from a few inches to 2 feet below their original level. In some cases the end of the crib next the lock wall has been bolted to the masonry, with a view to retarding this settlement. The practice, however, is of very little use, and results in unsightliness, as the end is held up while the rest of the crib settles, and the difference in level becomes more noticeable each year. It has always been found that where a timber guide crib or a dam of timber cribs is of any height, a settlement commences as soon as the work is finished, owing to the weight of the riprap filling gradually compressing the fibers of the wood, and the best that can be done is so to design the work that the settlement will be equalized as far as possible. A frequent cause of failure, especially where the cribs are high, lies in want of care in providing for this weight. Actually, a very large portion of the riprap is upheld by the timber, because the corners of the stone catch in the spaces between the ties or stringers. After a year or so the whole crib becomes choked with sediment, and the result is that an almost solid mass of masonry has to be carried by the ends of the ties, and this is where the first signs of failure are always shown. We have seen ties split open in new cribs through this pressure, and in old cribs they will be found to have become rotten at the ends, while the adjoining timbers show little decay. Relief may be obtained by placing blocks on each side of the ties to distribute the load, and care must be taken that the foundations are also equal to the weight to be upheld.

Where cribs have to be sunk through water to their foundations, as is usually the case where piles are not used, the same methods are employed as described for sinking

crib dams (Chap. IV), except that the base cribs should not be over 20 or 30 feet long, unless the water is shallow. It is more important to secure a thorough bedding than with a dam, since any settlement will cause the crib to lean. Where the foundation has been dredged out it is a good plan to raise the end of each base crib a foot or two, with the dredge or other power, and then let it fall, continuing the "shaking" until the timbers appear to have found a solid bed. The top can then be leveled up with shims or blocks at the water-line.

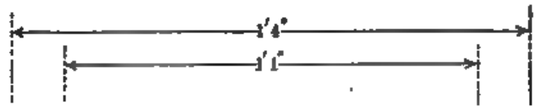
The cost of timber guide cribs with stone filling, when built by day labor and including all labor and material needed to complete them ready for use, varied in 1900 from two to three dollars per cubic yard of total contents, and from forty to fifty-five dollars per thousand feet B. M., on the basis of the timber. Owing to the increase in cost of labor and material since that time, however, the expense has increased from 50 to 100 per cent. As the life of a wooden crib is usually about 10 years for the portion above the water-line, and perhaps twice that time for the portion below water (owing to softening and wear of the timbers), it will be seen that they are not particularly economical.

Pile Guides.—Where the bottom is suitable, a cheap and effective guide may be made with a single row of piles driven 8 to 10 feet apart, and provided above the pool level with a grillage of 10×10-inch timbers, spaced 10 inches apart vertically, and bolted or drift-bolted to the piles, thus forming a continuous crib face. (Pl. 46*a* and p. 424.) Some of the piles may be left projecting above the top timber, to serve as check posts. The outer end should be provided with a short crib or a cluster of piles, as a buffer post. In cases where this method would cause trouble to craft because of the water drawing toward the dam through the open spaces, rectangular or triangular cribs 20 feet to 30 feet in length may be employed, spaced 20 to 30 feet apart, the openings being spanned by a grillage of timbers as for the piles. (See cuts, p. 424.) These types of guides will usually be found preferable to solid cribs, as the entrance does not so easily silt up, and they are less costly in establishment and repairs, while proving as satisfactory to boats.

A plain row of single piles, or of clusters of piles is not desirable for a guide wall unless the entrance is an unusually easy one, as the corners of barges catch against them, delaying the maneuvers, but for a guard wall, where economy must be considered, pile clusters answer very well.

Snubbing Posts, Line Hooks, Ladders, Gauges, etc.—For holding craft while locking, check or snubbing posts on the coping are required. They are as a rule made of cast iron, and are usually not less than 8 inches in diameter, and 12 to 18 inches high, and set from 50 to 75 feet apart along each wall. Cast-iron posts, when not filled solid or provided with drain holes, have been known to split through the freezing of water inside. On the Monongahela River locks where there is a large traffic of coal barges, steel pins $4\frac{1}{2}$ inches in diameter, projecting 12 inches above the wall are used. These

are preferred by the boatmen, as it is more easy to throw a line over them than over larger ones. They withstand satisfactorily the ice and the drift and occasional barges which sweep over the walls during floods. On the Ohio River locks near Pittsburg extra heavy 5-inch steam pipe is used, filled with concrete and projecting 10 inches above the coping. Fig. 166 shows the type of post used for lake steamships at the Sault Ste. Marie Canal in Michigan. This was evolved after several years of experiment, and has proved very satisfactory.



CAST IRON SNUBBING POST
WEIGHT ABOUT 300 LBS.

FIG. 166.

Snubbing posts should also be provided on the guide cribs or walls set about as described for the lock or a little further apart, so that craft can be tied to them while waiting their turn to lock. Those on river locks are usually set in the masonry about 4 feet or more back from the chamber face, depending on the location and on the width of coping. On canal locks where deep-draft vessels are handled and where there is ordinarily a wide back-fill of earth they are sometimes set from 12 to 20 feet from the face.

Another means of fastening is the line-hook. This consists of irons set into recesses in the chamber walls, usually about 50 feet apart horizontally and 8 to 12 feet apart

vertically, with the projecting ends shaped like hooks or bent into an L. The iron varies from $1\frac{1}{4}$ inch to 2 inches in diameter, according to service expected, and the part built in is forked to prevent the hook pulling loose. The face of the hook should be set back an inch or more from the face of the chamber, so that boats will not strike the iron. The recesses may be bowl-shaped or rectangular, and may be about 2 feet across the face and about 1 foot deep. With high guide walls, these hooks are very useful to small boats when approaching the lock and while waiting to make a lockage, as the lock tenders are then engaged and cannot take the line to a snubbing post.

One advantage of line hooks over snubbing posts is that the lines lead less vertically to the barges and can be held better on the timber heads or bitts.

On some locks cast-iron chocks are provided at the ends of the chamber, set close to the edge of the land wall. The mooring-lines are led through these to snubbing posts, and the chocks prevent the ropes from being chafed along the coping as the boat rises or falls in the lock pit. This is an excellent arrangement.

The ladders in the chamber should be of iron, and one should be set in the land wall just above the lower gate recess, and one in the river wall just below the lower gate so that the deck-hands can climb up or down as the boat enters or leaves the lock. In addition to these, one should be placed in the river and land walls, just below the upper gates. Where the chambers are very long intermediate ladders should be provided, and if the dam is a movable one, a ladder should be placed on the outside of the river wall, 30 or 40 feet above the crest. The ladder recesses are usually about 12 inches deep and 20 inches wide, with the center of the ladder set about 4 inches back from the chamber face, and a clearance of 2 inches or more between the line of this face and the face of the iron, so that boats will not strike the ladder. Sometimes the recesses are made about 20 inches in depth, so as to shield a man from passing craft. The ladders are usually removable, so that they can be replaced if rusted away or broken, and an excellent type consists of rungs of 1-inch diameter iron, about 1 foot long and 1 foot centers, with the ends turned down and set into uprights of $2\times2\times\frac{5}{16}$-inch angles. These angles are fastened to the wall by $\frac{3}{4}$-inch bolts set into the masonry about 5 feet apart vertically. On the top of this wall a hand-hold should be provided to assist getting on or off the ladder. This frequently consists of a U-shaped 1-inch iron rod, set into a recess in the coping, and with clearance around it sufficient to allow a man's hand to grasp it easily. A drain hole should be provided in the recess. Another and more serviceable kind of hand-hold—except that it is in the way of mooring lines—is an upright piece of $1\frac{1}{2}$-inch gas pipe 3 feet long, set loose in a socket in the coping.

A form of ladder sometimes used consists in U-shaped or straight iron or gas-pipe rungs embedded in the masonry of the sides of the recess, no frame being required. It has the objection, however, of being difficult of renewal when rusted.

Where traffic is heavy, two power capstans are sometimes provided, one being

usually placed just above the upper gate on the land side and one just below the lower gate. In other cases only one is provided, set at the middle of the lock, and with chocks at the edge of the wall for leading the lines. Their purpose is to facilitate the entrance of barges and craft which are without power. For boats up to about 1000 tons the capstan is usually made 10 horse-power or a little more. For notifying boats to approach the lock railroad semaphores are very useful.

Gauges to show the limiting depth of water at the lock should be set in the river wall (where they can be read from the land wall), one just above the upper gate and another just below the lower gate. They may be made of tile or may be cut or formed in the masonry, and should be set in not less than 1 inch to avoid being injured by boats. The numerals should be Arabic, and may be 4 or 5 inches in height, and the divisions are usually made into feet and tenths. If the masonry is to be of concrete, the form for the gauge can be conveniently made by taking a dressed 12×1-inch plank, beveling the edges, and tacking on the back the numerals and the divisions. These can be cut out of wood and should be V-shaped so as not to adhere to the concrete, and about $\frac{1}{2}$ inch wide on their faces. They may be made $4\frac{1}{2}$ inches long for the foot divisions, 4 inches for the half-foot, and 3 inches for the tenths. This method, if care is applied, provides a gauge of very neat appearance.

Tile gauges of mosaic are now being used in many instances. The tile is of the character known to the trade as ceramic mosaic and is non-absorbent to water, grease or oil. The gauges are usually made up in panels 12×18 inches temporarily mounted on paper. The pieces of which the panels are composed are $\frac{1}{4}$ inch in thickness and generally $\frac{3}{4} \times \frac{3}{4}$ inch on the face. On the Panama Canal over 5000 square feet were used, varying in face measurements from $\frac{3}{4}$ to $1\frac{1}{8}$ inches square. The gauges are generally made in two colors, a white ground with navy-blue figures and graduation marks being a favorite. The gauges for the Panama Canal cost 28 cents per square foot, delivered and ready for erection. A common method of setting the tile is to back the panels with cement mortar $\frac{1}{2}$ inch in thickness, in which are placed projecting wire staples. Another half-inch of mortar is then spread on the wall and the tiling set against it, the staples giving a firm hold between the two layers.

Finish of Coping.—The inside edges of the coping of the land and river walls should be rounded off to a radius of about 2 inches, to prevent chafing of mooring-lines. It is good practice, in fact, to round off all coping edges, as they are liable otherwise to become chipped and disfigured.

On concrete locks this radius can be made best by using a quarter-round edging trowel, running it along just inside the form when the masonry has taken its initial set. Another method is to use a concave moulding of wood or sheet metal. The former can usually be obtained from planing mills. Care must be exercised to see that the stone in the concrete is properly worked back from the face, or it will produce a ragged appearance. This is less liable to occur

with the troweled edge, as any irregularities can be seen and remedied at once. A radius of 2 inches has proved suited to locks for craft up to two or three thousand tons, but for ship locks a radius of 6 inches is often used, as it is easier on the steel hawsers.

The tops of the walls should be crowned or sloped about $\frac{1}{2}$ inch for drainage, or they can be given a uniform slope from back to front of $\frac{1}{8}$ to $\frac{1}{4}$ inch to the foot, as with sidewalks.

CONCRETE LOCKS.*

General Design.—(See also pp. 393 and after.) The use of concrete for river works was practically untried in the United States until 1892, although it had been used for other hydraulic works for many years. So far only two objections have been made to it, one, that its appearance is inferior to that of a lock of cut stone, the other, that it may not prove durable. However, some concrete locks were built about 1795 on the Franz-Josef Canal in Hungary and were still in satisfactory use a century later, although naturally considerably disfigured by age and use. The latter objection can only be answered by experience, but there seems no reason why concrete should not last as well at least as the softer classes of stone, since the latter are considerably affected by water and by weather. Another objection sometimes urged is that a concrete wall, being divided into sections by vertical joints, is less strong than a wall where each stone is bonded, but where the foundations have been properly designed this objection appears to have no appreciable effect in practice.

Concrete is generally used where its cost would be less than that of cut stone. The materials are usually obtainable near the site and are easily handled, and require no skilled labor in placing. The last factor is frequently one of importance, since masons and stone-cutters are often hard to obtain, and when obtained cannot always be depended on. A concrete wall can also be built much more rapidly than one of stone.

The design for a lock of this class should provide outlines as simple as possible, and offsets, curved surfaces, and difficult intersections (which are rarely necessary unless special effects are desired) either in the walls or in the culverts, should be avoided as far as practicable. This is desirable because it lessens the expense of the forms, and also permits them to be set up more rapidly, and a good deal of delay and extra work may be caused by having to stop concreting in order to change or set up forms for changes of surface. Surfaces curved in one plane only, as the end of a wall, are not specially objectionable, but forms curved in more than one plane, such as the circular elbows sometimes met with in culverts, are very expensive to build and difficult to keep in shape when built. The number of bolts, castings, or other parts requiring to be set during construction should be reduced to a minimum, as it is more difficult to place them

* For a description of the rubble-and-grout lock walls of the Nile Delta see Chapter IV, under the heading "Weirs on the Nile."

accurately in a concrete wall than in a wall of stone, owing to the difficulty of holding them in place while working. Square corners on exposed surfaces should be avoided, as they are easily chipped off, and corners which are liable to be struck by boats should have a radius or bevel of not less than 2 feet.

In regard to using stepped or sloping backs for walls to be back-filled with earth, some engineers prefer the former, on the ground that greater adhesion is obtained between the masonry and the earth, while others prefer the latter on the ground that the forms are easier to build. The difference in the cost of the forms, however, is in fact small. If steps are used, they should be not less than 3 or 4 feet in height, as such are easier to provide for than low steps.

The walls should be divided into sections not much over 25 to 35 feet in length, otherwise they may crack open near the center of the section with shrinkage in drying and with temperature. The opening of the joints will also become very noticeable in cold weather. In Minnesota sections have been used only 16 feet in length. Where long walls have been built in one length such cracks have appeared at intervals of 25 to 50 feet, and even where sections 45 feet long have been used a slight crack has usually appeared close to the center. While such partings are no more weakening than artificial joints, they are far more unsightly. The joints made during construction, when exposed to a considerable head of water, usually leak for some time after construction, as may also the main wall itself, especially if put in as "dry" concrete, but the action of the water on the carbonates in the cement, and the finer sediment of the river, will gradually seal them up. There is, as a rule, no need to use any separating material, as tar-paper or paint, in the joints; satisfactory results are obtained by placing the new concrete against the old unless a rich concrete is used, when a coating of oil or of soap may be advisable. A concrete of 1 part of cement to $7\frac{1}{2}$ parts of aggregate has been known to adhere at the joints occasionally, while elsewhere with the same mixture no adhesion occurred. Similar sections should be made of equal length where practicable, for reasons explained on the paragraph in "Forms."

Where the natural foundation can be easily drained, so that trouble from leakage or from caving material can be overcome without difficulty, the main outlines of the walls may be designed to start from the bottom, without offsets, unless the toe-pressures require the latter. Where, however, this condition is absent, as is usually the case, it is an excellent plan to provide a footing course a few feet high, and a foot wider all around than the main body of the walls. Its top will provide a platform above the leakage, and the forms can be set up without fear of displacement from caving excavation, and with the leisure necessary for accurately lining them in.

A general plan and section of one of a series of concrete locks of recent construction is shown on Pl. 45*b*. This lock possesses the unusual feature of duplicate sets of culvert valves.

Proportions and Materials.—The proportions of the mixture in the earlier locks were considerably richer than those in later ones. On the Illinois and Mississippi Canal (1894) they were one part of Portland cement to seven or eight parts of sand and gravel or broken stone On Locks Nos. 1 and 2, Big Sandy River, Ky., and W. Va. (1902), they were one part of Portland cement, three parts of sand, and six parts of mixed gravel or mixed broken stone; while on Lock No. 9, Kentucky River, Ky. (1902), they were one barrel of Portland cement, 15 cubic feet of common river sand, and $33\frac{1}{3}$ cubic feet of mixed broken stone, from $\frac{1}{2}$ to $2\frac{1}{2}$ inches in diameter, giving a mixture of about 1 to 12. The last mixture possessed when hardened an abundance of strength, as was found when drilling into it for setting bolts. It appeared in fact to be about as hard as the mixture used for the Big Sandy locks. The foundation of the Mirowitz Bridge Dam on the Moldau (1902) was composed of 1 part of cement to 10 of sand and gravel; and the foundation of the Assiout Bridge Dam on the Nile was 1 part of cement, 3 of sand, and $6\frac{1}{2}$ of stone. Lock and Dam No. 5 on the Monongahela River (1910) were built of a 1 to 3 to 6 mixture, and similar proportions were then in use on some other of the tributaries of the Ohio.

At Lock No. 2, on the Mississippi, near St. Paul (1900), "sand cement" was used instead of pure cement, and consisted of a mixture of one part of cement to one part of sand, ground up together very fine. One part of the resulting product was then mixed with approximately 3 parts of sand and 6 parts of stone, the proportions thus being 1 to 19 on the basis of the cement alone. The grinding plant was owned and operated by the United States.*

A striking example of the strength of concrete masonry is to be found in the entrance piers of the Duluth ship canal on Lake Superior, where 1 part of Portland cement was used to 3.4 parts of sand and 6.7 parts of pebbles. These piers, built in 1900–1901 and exposed to unusual conditions of storm and ice and to very severe usage from impact of the large lake vessels resulting from storms or from careless handling, have stood all tests practically uninjured. From this and similar instances it would appear that these proportions where adopted should be satisfactory for any class of lock masonry.

All the above-named proportions were by measure, except that in some cases the cement was taken as packed, in others as loose measure. Specifications should always state whether the cement is to be measured loose or packed, loose measurement being the volume when the product is poured out of the bag or barrel. The contents of a barrel when shipped vary in general with different brands from about 3.03 cubic feet to about 3.35 cubic feet, and when loose from about 3.75 cubic feet to about 4.19 cubic feet, according to the fineness of the grinding. Tests made from a number of different brands gave an average of about 3.18 cubic feet in the barrel, and about 4.07 cubic feet loose, the ratio thus being 1 to 1.28.

Sometimes a natural cement has been used for the parts below water, for the sake of

* Annual Reports, Chief of Engineers, U. S. A., 1900, and after.

economy, but Portland cement should always be used above, as it stands wear and exposure much better, and is more carefully manufactured.

The sand should be clean, and preferably of coarse and fine grains mixed, as a sand of coarse grains only will leave too many voids in the mortar. A good average quality will be obtained by having it fill the following specifications for fineness: To pass a No. 30 sieve, not over 70 per cent; to pass a No. 50 sieve, not over 50 per cent; to pass a No. 100 sieve, not over 2 per cent.* It should always be tested in briquettes, as some sands contain foreign particles apparently sound, which nevertheless have a weakening effect upon the mortar. It is often impracticable to secure sand or stone that can be made entirely clean, and in such cases a small amount of earth or foreign matter is not now considered a drawback, as it has no appreciable effect in the mass of concrete.† Some specifications limit this amount to 2 per cent, while others (as those for the New York State Barge Canal) admit 5 per cent without washing. Similarly the gravel will usually retain more or less sand after screening, the variation in the amount being usually limited to 6 per cent. The total amount of earth, etc., in the gravel and sand when combined, or in the sand and broken stone, should also be limited to about 6 per cent, but should be as much less as possible, because it produces a large amount of "laitance" or slush in the concrete which is troublesome to get rid of and may produce weak spots in the walls. If crusher dust is used in the concrete, tests should be made of the amount of fine material, as some stone will produce so much of it that it has the same effect on the concrete as an excess of loam or silt, and the masonry when set will be "soft." Where tests show an excess, the fine material should be rejected and an equal amount of sand used instead.

The stone may be either broken stone or gravel, and should be free from dirt. Many engineers prefer gravel, as it is more easily handled, and gives a denser concrete than the ordinary broken stone. For the latter limestone is considered best, as it appears in course of time to combine chemically with the cement, but any good hard stone will answer. If the stone is soft, it is liable to crush under the rammers if "dry" concrete is used, besides being deficient in tensile strength. Certain classes of sandstone, however (especially those impregnated with iron), which are soft when quarried, will become reasonably hard after some weeks' exposure to the weather, and can then be used, where better stone is not obtainable, with good results. On one lock on the Big Sandy River stone of this character was used under conditions where it was not practicable to

* In certain localities it has not been possible to secure a coarse sand without great expense, and in such case common fine-grained river or bank sand has been used, and has given results amply strong for all practical purposes.

† Experiments with some briquettes composed of 1 part of Portland cement to 1 part of sand and river mud, and with others composed of 1 part Portland cement to 3 parts of sand and river mud, showed that where the mud did not exceed 5 per cent of the amount of sand no appreciable diminution of strength occurred, and that it was not until the amount exceeded 10 per cent that any marked reduction was observed. Comprehensive tests by various experimenters were reported in Engineering News, November 21, 1903, and later. The chief objection to the foreign matter is the slush or "laitance" it produces in the concrete (unless "dry" concrete is used) as noted further on.

season it, and much of it when used could be broken by the fingers. The resulting masonry, however, during an exposure of several years, has proved entirely satisfactory. The stone or gravel is usually limited to 2 or 2½ inches as the maximum diameter, and includes all smaller pieces down to ⅛ or 1/10 of an inch in diameter. A maximum diameter of 1½ to 2 inches, if broken stone is used, is preferable, as it gives a concrete with fewer voids. In some cases the material from the crusher has been used without any screening (it is then called "run of crusher") with satisfactory results, provided it did not contain much dust.

If "dry" concrete is to be used (see p. 441), the stone should be thoroughly wetted before being put in the mixer, and the sand should be dry, or not more than moist as when it is wet it requires more mixing to incorporate it thoroughly with the cement, and the varying amount of water in it will cause a constant variation in the wetness or dryness of the concrete. If "wet" concrete is to be used there is no need to moisten the stone, and the amount of moisture in the sand is of minor importance.

Of slow-setting and quick-setting cements the former is preferable, as with the latter, if delays occur, the concrete may be spoiled, besides which, the workmen in the forms keep the surface more or less disturbed, and if it has begun to set before the next layer has covered it, the bond will be destroyed. A cement which will not take an initial set in less than an hour, at a temperature of 70° Fahrenheit, is well suited for this class of work.

The use of different qualities of cement, or different proportions of the same cement, in the same section of a wall, is not a satisfactory practice from the point of view of construction. It has been done sometimes to save expense, a Portland cement being used for the outside 2 feet or so of thickness, and a natural cement for the interior of the walls, or a richer and poorer mixture respectively of the same cement has been used. In the former case it requires a constant attention from the cement house till the concrete is in place, in order to see that the proper cement is put into the mixer when called for, and in either case careful watch must be kept to see that when mixed it is placed in its special zone in the wall, and even with the best of care the batches will sometimes be misplaced, leading to delay. It appears certain, moreover, that as Portland and natural cements usually have different speeds of setting, the bond between them is not as strong as is desirable. In certain walls built with such combined concrete a parting between the two kinds has actually occurred.

The practice of embedding stones in the walls for the purpose of saving cement is not always an economical one, unless they can be set without causing delays in the concreting. Some specifications limit the size of such stone to 1 cubic foot or over, but there appears to be no valid reason against using smaller stone, since the concrete itself is an aggregate of small pieces. Walls with stone properly embedded have been found to give as satisfactory results as walls of solid concrete.

Forms.—The term "forms" is given to the timberwork set up as moulds to hold the concrete until it has set or become hard enough to stand alone. The "lagging" is the plank against which the concrete rests. The general design of forms consists of such plank laid horizontally one on another, and supported at the necessary intervals by vertical posts which in turn are held in place by braces or struts, or by a combination of ties and struts. In building the earlier concrete locks the forms, after being used for one section, were generally taken down piece by piece and set up similarly for another section. This method involved much rehandling and repetition of carpentry work, but under existing practice the parts composing a form are usually fastened securely together in large panels and made of the full height of the sections (varying from courses a few feet in thickness to the full height of the wall) which it is intended to build. Wherever conditions permit, however, the forms should be made of the full height of the wall, so that when set up concreting can be carried on course by course until the block is finished. While this takes more lumber than building the block in forms of say half the full height, and then moving these up and resetting them, considerably less labor is required and much time is saved by using forms which will need no moving until the entire block is finished. This method also avoids the scarring of the edges of the masonry, almost unavoidable if the forms have to be shifted. When ready for removal the sides and ends of the form are disconnected and the pieces are lifted out by a derrick and set in their new positions, and only require bracing and tieing to be ready for use. For this reason the various parts of the walls should be made similar as far as practicable, as one form can then serve for all sections which are alike.

The principal desiderata in forms are a smooth inside face, and general stiffness in design and bracing, so that they will not easily get out of line during concreting. Much lighter timbers are now used than in the first locks, where the posts were 8 by 10 inches, 4 feet apart and each with its own set of braces, and the lagging was 4 inches thick. For the same purpose posts are now used 3 by 8 inches, $4\frac{1}{2}$ feet or more apart, with lagging 2 inches thick and braces 10 feet apart, and the results are equally satisfactory. This is partly due to the change from dry to wet concrete, and partly to a more extended experience. The constant ramming needed with dry concrete had a powerful effect in springing the forms, and these in consequence had to be extremely stiff; with wet concrete this effect is absent, although the liquid pressure appears to be much greater. It is rare, however, that a section is built up rapidly enough to create a greater head than 5 or 6 feet; by the time that pressure is reached the under portion of the mass has usually begun to set and so causes little further thrust. Fig. 167 shows a design which has given good results in practice, and is typical of present methods. It consists of lagging nailed to uprights which in turn are fastened to stiff longitudinal timbers or walings. The latter are held apart at the proper distance across the form by tie-rods, or better still by tie-rods and struts, and should be also strongly braced on the outside at these points if practicable. Where there is a high bank on each side of the form braces can be used

on each side so as to prevent any swaying; where the bank is on one side braces are usually placed on that side only, and guys of wire rope, adjusted by long eye-bolts, are fastened at one end to the form where the braces occur and to "deadmen" in the bank at the other. By tightening the eye-bolts the ties and braces in combination hold the form in place. Special care should be used in supporting the braces against the earth, as they are very liable to slip or give way there unless the bank is solid and the footing pieces broad and well bedded. The guys should not be long, because of changes due to temperature. In some instances the braces have been omitted and guys used on both sides instead. This method is useful for high walls where very long braces would be required, but as the guys are apt to stretch they need constant watching and adjustment. In other instances all guys and braces have been omitted, but this is not desirable, as the forms get out of line too easily. Still another method, which has proved

Fig. 167.—General Framing of Form with Outside Braces.

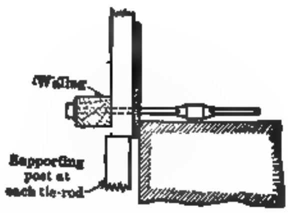


Fig. 168.—Method of Holding Lower Part of Form at Intermediate Levels.

very serviceable with forms of small height, is to use an X-bracing of guys or tie-rods as at each end, adjusted by turnbuckles so as to hold the form to line.

The tie-rods across the forms should be not closer than 10 feet if practicable, as they are much in the way of the buckets carrying the concrete, and are frequently struck and bent in consequence. If the forms are removed within two or three days after concreting the rods can usually be pulled out without great effort, but a better way is to provide sleevenuts (usually made of steam pipe) about a foot from each face, and to grease or wrap in paper these ends of the rods (Fig. 168). When the forms are removed the sleevenuts and center portions of the rods are left in the wall (the end portions being unscrewed and pulled out) and the holes are plugged with mortar. Another method omits sleevenuts and uses tin pipe around the rods. All the rods can thus be pulled out, the pipe being left in place. However, if the rods are not straight, it is troublesome to remove them, and the method of sleevenuts is the easier. Still another method is to twist the rods occasionally while the concrete is setting.

Whatever type of form is adopted, and however carefully they are braced, the forms must be closely watched for line during concreting, and a very convenient way to do this is to set a row of hubs in the ground, 10 feet or less apart, close to and parallel to the line of the forms, with center points in them. By hanging a plumb-bob from the top of the timber-work over each point any displacement can be immediately seen. The forms for walls with one sloping side are usually more difficult to hold to line than are rectangular walls.

All lagging plank, where a good surface is desired, should be seasoned and dressed and preferably have matched edges to prevent leakage of mortar, and the edges of the posts should also be dressed, so as to secure an even line. The additional cost is small, and unless it is done bad joints and irregular surfaces will be the result. Immediately after use lagging should be well cleaned if it is to be used again. Yellow pine appears to be more satisfactory for form lumber than other kinds of timber, as it is less affected by the moisture and less liable to warp and shrink, but if the forms are well made and properly braced and looked after the kind of lumber becomes a matter of minor importance. Joints between the lagging can be temporarily closed with oakum, or with stiff clay rubbed in to present a smooth surface. In one or two cases the lagging has been lined with galvanized or plain sheet iron, well greased, and this is said to have given an excellent finish, doing away with the joint and grain prints which are inseparable from wooden lagging, and which have caused, on the score of appearance, one of the principal objections urged against concrete locks. However, the later method of rubbing down the face, described further on, has eliminated most of the difficulties met with in this respect in the earlier construction.

The forms for the culverts and other inside openings are usually made of rough plank, of the same thickness as the outside lagging and with stiffening frames about 4 feet apart, or 1 inch lagging can be used with frames about $2\frac{1}{2}$ feet apart. It is very advantageous to design these forms so they can be taken apart and removed in small sections, thus allowing them to be used over again without knocking them to pieces.

Sloping upper surfaces, such as the coping of the tail wall of an abutment, have to be planked over on the top if wet concrete is used, since this material will begin to flow under a very slight inclination.

A good example of modern practice with forms may be found in the construction of the Barge Canal of New York State, 1905 and after, where more than sixty locks were built of concrete, with walls ranging up to more than 60 feet in height, besides a large number of retaining walls, piers, etc., the total amount of concrete used being nearly three million yards. The method above described of building the forms in permanently fastened sections and then using these sections as many times as practicable, was followed almost entirely. The heights of the sections varied from a few feet to the full height of the walls, where the latter were not over 20 or 30 feet above the foundations, the higher sections usually being the more economical in handling and in time.

View of Concrete Lock with Inverted Floor. The chamber is 45 feet wide with a depth on the upper miter-sill of 16 feet and a lift of 15 feet. The floor is 3 feet thick, with piles embedded in the concrete. (Cranesville, Mohawk River, N. Y., 1909.)

Building Concrete Lock Walls by the Method of Sections, using Braced and Cantilever Forms. (New York State Barge Canal, 1907.)

Where the walls were high and outside bracing would have been costly, some contractors used sections only a few feet in height, holding the tops in place by tie-rods (Fig. 169). The lagging was usually 2×12 inches, dressed on one side and with matched edges, as these prevented leakage through the joints. The best results were obtained where the forms were braced on all outside faces and had tie-rods and struts to keep them at the proper distance apart inside, although good results were also obtained where braces were used on one side only and held back by ties as previously described. This method, however, was found to require more constant attention in order to keep the forms to line. One contractor, building a lock set into rock, fastened his face forms to a heavy trestlework moving on a track along the lock chamber, and shifted this support from place to place, adjusting the forms by screw rods, so that no other bracing was required. Cases where special methods were used, such as this one, were, however, very exceptional.

If the lock is to be built by contract it is often preferable to have the general design of the forms carefully worked out, and the drawings and specifications

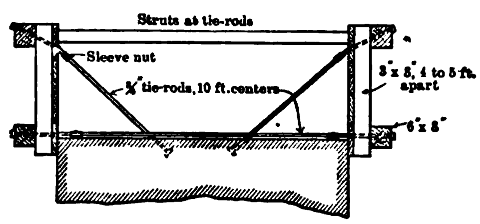


FIG. 169.—Cantilever Form—no Outside Supports. Often used for high walls, the ends of the form being held in position by an X-bracing of tie-rods. In some cases the struts and slanting tie-rods have been omitted, and an end X-bracing of rods used instead, fastened to the projecting ends of the walings.

for them made a part of the agreement. Few contractors have had much experience in building concrete locks, and their knowledge of what is required for satisfactory forms is consequently limited; while, on the other hand, the collective experience of constructing engineers is large and should be sufficient to permit of obtaining the best results. If the design is left to the contractor he will naturally endeavor to use cheap methods and materials, whereas to secure a form that will give straight surfaces and a good finish to the walls a considerable expense must be incurred, and unless suitable designs are exhibited to the bidders as a part of the contract, friction and dissatisfaction may later result. Nothing is more annoying in this respect, either to engineer or to contractor, than to have during concreting a constant lining-up of forms and lagging which have got out of place because of want of stiffness in the posts or for lack of proper bracing. In some cases it has taken the entire time of an inspector, besides the time of employees of the contractor, to watch and correct changes in this portion of the work. Once the form has been set up and brought to line it should need little more attention, and where proper experience has been brought to the design this result can be secured with but small extra expense.

In recent construction specially designed steel forms have occasionally been used with satisfaction, and if these forms can be used many times they frequently prove to be cheaper than forms of timber. Sometimes the faces are oiled to prevent adhesion to the concrete, but in most cases no coating is applied.

Mixing and Placing.—In mixing the concrete, which should always be done by machinery if practicable, the materials are usually dumped into the box or into the hopper all together, the water poured in on top, and the charge mixed. In one type of rotary mixer the water is introduced after the box has commenced to revolve, the shaft being made hollow and pierced with holes, and connected by a pipe with the water-tank. While this method is preferable in theory, it has drawbacks in practice, as the holes become clogged with mortar or spalls, resulting in an uneven mixture, and a necessity for cleaning them out every few hours. Fewer turns are required if the materials are first revolved dry and the water then introduced, but this is rarely done in practice, and equally good results are obtained by the method usually followed. Gravity mixers, of which there are several types, mix the ingredients by causing them to fall from a height into one hopper after another, or to become mingled by striking obstructions as they fall. Some of these mixers are capable of an enormous output, much greater than can usually be taken care of in lock work. While the resulting mixture is in general satisfactory, the process cannot be controlled as well as in the rotary mixer, and if carelessly tended there is no way to incorporate ingredients more thoroughly except by passing them through the mixer again or by turning the mass over with shovels. However, if close inspection can be secured to guard against the laborers' inattention (which is the usual trouble with this type of mixer) the general results should be satisfactory. From eight to sixteen turns are usually required for rotary mixers, depending on the size and type. No special order need be observed in putting in the materials, except that the water is usually put in last, nor is it necessary to mix first the sand and cement.

When the concrete is mixed it is taken to the forms by dump-cars, or in dump-boxes handled by derricks. Frequently a track is laid in the lock pit on which a traveling derrick runs, taking the boxes from stationary derricks or from cars, and moving back and forth as required, and experience has shown this to be one of the most convenient methods of handling. Where the concrete has been carried in the cars themselves, they have sometimes been run on a track laid over the posts, but placed far enough to one side to allow dumping into the forms below. This method is objected to by some engineers on the ground that the dumping tends to separate the materials, but it has been used in several locks where the fall was from 25 to 35 feet, and no trouble of this nature was experienced. The spreading of the concrete with shovels will assist in correcting any tendency to separation should it become apparent. A more serious objection is that the fall of the mass and the movement of the cars shakes the forms, however well they may be braced, and for this reason the concrete, if dumped from any

considerable height, should be slid down a chute, when it will reach the bottom gently, and the supports for the track should be kept entirely separate from the posts of the forms. It is stated that with chutes lined with galvanized iron wet concrete will begin to slide under an inclination of about 15° in warm weather and 30° in cold weather. A somewhat steeper inclination, however, is desirable for ease of flow.

After the concrete is in the forms it should be spread out in layers 6 or 8 inches deep and be compacted. The ramming, if dry concrete is used, is done with cast-iron rammers, about 6 inches square on the face, and weighing with the handles from 18 to 30 pounds, according to the ideas of the engineer in charge, the usual experience being that where laborers are kept at ramming all day considerably better results are obtained by using the lighter weights. If wet concrete is used the shoveling and the tramping of the men over the surface will compact it satisfactorily.

For setting valve seats or other heavy ironwork, the concrete should be built up to form a base and be allowed to set for a day or two, as it is very difficult to support and hold to line heavy pieces if the concrete is not set. Light pieces may be supported on unset concrete by using plank which can be built in and left there, but care must be taken with all ironwork to hold it securely, and it must be watched until safely embedded, as it is liable to settlement as well as to be disturbed by the laborers. If the pieces require accurate adjustment, recesses should be left in the concrete and the pieces should be set after the concrete has hardened. They can then be held in place by blocking and wooden wedges, which can be left in the concrete. Such recesses can be built with dovetailed or keyed sides, so that the new concrete will be bonded firmly to the old. Bolts for ladder supports, frames, etc., can be built in by boring holes in the lagging for them and letting them project inside to the desired amounts. (For joints and facing see pp. 443 and 444).

After a section has been finished it should be covered with moist canvas or sacking for about two days, or better still with about an inch of clean, wet sand until the coping is well set, and if dry concrete has been used, the whole mass should be wetted twice a day for two or three days, or more if the weather is warm, as this class of concrete is very thirsty. If wet concrete has been used, the coping only will need moistening. (See also p. 445.)

The forms can be removed in from two to five days, depending chiefly on the conditions of the temperature, the longer period being advisable in cold weather, as the concrete hardens then more slowly. If the sections are of small height, there is no danger in removing the forms sooner.

Dry and Wet Concrete.—" Dry " concrete is concrete in which only enough water is used to enable the cement to coat the aggregate; " wet " concrete is provided with sufficient water to reduce it to the consistency of semi-liquid mud. Dry concrete was used in practically all the earlier work built in America, chiefly because laboratory experiments showed it to be somewhat stronger than wet concrete. It required in the field, however, a vast amount of care if specifications were to be complied with. The

amount of water had to be gauged exactly, as the requirements usually called for the concrete to show only a film of moisture on the surface after protracted ramming. This necessitated the wetting of the broken stone and of the lagging, so that they would not absorb any of the water from the mixture, and if the sand happened to vary in its moisture, allowance had to be made accordingly. The temperature also affected the mixture, the difference in the required amounts of water between early morning and mid-day sometimes reaching 25 per cent. In addition to this, a special mortar facing, at least 1 inch in thickness, had to be used for exposed surfaces (as described on p. 444), and constant vigilance was needed to see that the concrete was properly compacted and did not dry out on its surface before the next batch was laid. After the completion of a section it had to be kept covered and be supplied with an abundance of water for several days so as to secure a proper setting, since concrete which becomes dry before it has set properly loses most all of its cohesion.

It was found by experience that this class of work was usually porous and, contrary to the accepted belief, did not posses the strength of a wall of wet concrete, and to-day the use of dry concrete has been practically abandoned.

The amount of water for wet concrete should be enough to make it flow like a semi-liquid mud. In this condition it can be easily shoveled and spread, and requires no tamping, as the men walking on it when shoveling compact the mass thoroughly. No special mortar is used for facing as with dry concrete, but a narrow spade or a fork with tines close together is pushed down all along the lagging, and the stone worked back from the face, the mortar of the mixture flowing to the front to replace it. This gives a very satisfactory facing, and one which is in thorough union with the mass behind. One point has to be watched carefully, however, with wet concrete. If the amount of water is considerable the "laitance" or fine soapy inert matter in the cement as well as the earthy matter in the aggregate, will be worked to the surface as the men walk on the concrete, and at the end of the day's work will settle on the top, sometimes to a depth of 1 or 2 inches, causing a weak stratum there. This material resembles chalk in texture and weight, and though appearing to inexperienced eyes as the best part of the mortar, it will be found on examination to possess little strength, and when exposed to the weather will gradually disintegrate. It is therefore very objectionable if it appears on the face of the wall, and to avoid any trouble the concrete should be kept high along the edges, so the water will drain away from them. This laitance should always be picked or scraped off any surface on which more concrete is to be placed. If the amount of water in the mixture is enough to produce this laitance its quantity should be reduced, or the excess water should be bailed from the surface or be allowed to run off through holes bored in the forms, even though a small amount of active cement may be lost thereby.

Laying Concrete under Water.—In parts of the foundations where leaks or springs are active, as in deep trenches, it is often advisable to let the water rise therein until it

becomes quiescent, before placing the concrete. In such cases, if the depth of water is only 2 or 3 feet, the concrete is placed first in one corner until it rises above the water line, and the remaining concrete is then dumped on and near the edge of that just placed, so that the mass will flow outwards slowly and drive the water before it. Usually very little cement is washed from the mixture during this procedure, and not in sufficient quantity to produce any weakness. In greater depths of water a similar process is followed except that the concrete is lowered through a canvas or plank tube, the outlet of which is kept close enough to the bottom to prevent the mixture from flowing out except when the tube is moved. By maneuvering the tube the concrete can be made to flow out and distribute itself slowly as in the first process. Concrete deposited in this way on the Mohawk River, N. Y., in 9 feet of water and cut into later showed a very satisfactory quality. In the Detroit Tunnel nearly all the concrete was placed in a great depth of water.

Much of the foundation of Lock No. 1 on the Allegheny River was laid under water, as it was found difficult to unwater the cofferdam. (See also p. 412.)

Monolithic Concrete.—In some locks the concrete has been built in courses a few feet in height, placed at different times; in others it has been built in monolithic blocks of the full height of the wall, and from 20 to 30 feet or more in length. The former method, as shown where the concrete has been removed, does not possess the strength of the latter (as each horizontal joint forms a plane of weakness), and requires in addition a constant moving of men and appliances from one point to another. The latter method, however, requires that work on the block must be continuous until it is completed, and this is often very difficult to carry out, so that monolithic concrete is not often employed, especially as it has been found that for all purposes of practical use results equally good can be obtained by working in daytime only. With the system of building in separate courses, moreover, nightwork can be avoided, an arrangement which is always desirable, as it is very difficult to inspect concrete properly at night, and in addition forms sometimes become displaced or out of line without the movement being noticed. Where special strength is desirable in the horizontal joints a mortar of neat cement, $\frac{1}{4}$ to $\frac{1}{2}$ inch in thickness, is often used. Laboratory experiments have shown that blocks of concrete thus joined will break almost invariably in the mass and not at the joint. The surface of each course is left in a ridge or a hollow along the center, or rough projecting stones are embedded in it, and it is important that the face edges should always be carefully leveled at the end of the day's work, so that will be no ragged or wavy joints or wire edges. A crooked horizontal joint, where above water, will always be noticeable after the wall is finished. On resuming work next day the top of the concrete is wetted, all the laitance is picked or scraped off, and a small amount of cement is scattered over and brushed in, thus forming a thin grout. These joints seldom show appreciably in the finished work if proper care is taken when being made. If a mortar

joint of specified thickness is employed, as described above, the mortar should not extend to the face, unless it is made of facing material, or it will show noticeably after exposure to the weather..

Each section of the wall should be well bonded vertically into the adjacent ones, not only for stability, but also because the tendency to leakage through the joint is thus reduced. In the first locks this bonding was made by placing vertically one, and sometimes two timbers, about 8 inches wide and 4 inches thick, along each end of the section being built. When the forms were removed the timbers were taken out, leaving grooves into which the concrete of the next section bonded. It has been observed, however, that any settlement of the walls almost always breaks off these tongues, and a better method is to take V-shaped troughs, which may be made of 1 × 12-inch lumber, and set them on end, either close together or with a foot or more between, the end ones being kept a foot or two from the face of the wall. These will provide a large number of bonds without any weakening of the masonry, and will hold the sections together against unequal lateral movement, besides being easy to place and remove.

Facing and Coping.—The walls should receive a facing wherever they will be exposed to view, or the stone or gravel will show in places and give the masonry a rough and unfinished look. Where dry concrete is used a special facing of mortar, usually a 1 to 2 or a 1 to $2\frac{1}{2}$ mixture, has to be employed. It is placed by taking planks 6 or 8 inches wide and 4 to 6 feet long, beveled from about 2 inches in thickness on one edge to 1 inch on the other, and placing them against the lagging. The concrete is spread and rammed against these plank, and the plank are then pulled up and the wedge-shaped recesses left by them are filled with well-rammed mortar, placed in layers each not more than 3 inches deep. More frequently a plain 1-inch board is used instead of a beveled one, with wedged-shaped galvanized iron ribs between it and the lagging. The mortar is packed between these ribs. The sand for the mortar should be of a fine rather than of a coarse quality.

With wet concrete a much simpler method is employed. The concrete is shoveled against the lagging, and a spade or a fork with close tines is worked down along the boards, pushing back the stone and letting the mortar of the mixture run in to replace it. One of the best implements for this work is a spade with narrow slits cut in it; wide slits will let the small gravel or stone slip through, when they are apt to show on the face. The only special attention needed for this method is to see that all the face is well spaded, and as it is quite inexpensive it is often used along culvert sides, the backs of walls, and other surfaces which will nevertheless not be permanently exposed. This method of facing has the objection, however, that when the walls are scarred by boats the broken stone or gravel of the matrix becomes exposed, making a faulty appearance. (See also p. 442.)

The raggedness of the vertical joints of the blocks, where the wall is built in sections, may be avoided by the use of triangular strips of wood, of cross-section like

a 45-degree triangle and measuring about ⅜ of an inch on the sides. One of these is placed in each corner of the block as it is built up. When the adjoining block is built up, or when the first block is continued, similar strips are placed, and the result shows, when the forms are removed, as V-shaped grooves of smooth outline. These should be continued across the top of the wall, or the parting will show ragged. Care is of course needed in making and setting the strips, so as to have them all of one size and placed truly. On some locks these strips have been used also at the horizontal joints between the various courses, and on other locks they have been placed so as to imitate the joints of cut-stone masonry. The practice, however, has not come into general use, although it produces a good effect.

As soon as the forms are removed men may be set to work to rub off the marks of the lagging joints, etc. For this purpose the various parts of the surface are kept wet and well rubbed with a flat stone or a carborundum or a cement mortar brick. If the lagging joints are prominent it is very difficult to efface them, but the rubbing will add very materially to the appearance of the wall, while if the joints are slight a smooth surface can be obtained with small expense. The work should be begun as soon as the forms are removed and before the sun and air have hardened the concrete, otherwise the amount of labor needed will be considerably increased. If the rubbing is begun at once a good laborer should cover about 250 square feet of wall in eight hours, the joints being those of 12-inch plank.

No plastering or cement wash should be allowed in climates subject to frost, or the surface will surely crack and come loose in course of time. If occasional holes or breaks have to be treated, the edges should be undercut and mortar fillings used of the same mixture as that of the concrete.

Where the forms for a wall are made of small height so that they require shifting once or more in building up a block (see cut on p. 438), the top edges of the courses are very apt to get chipped and scarred in moving and resetting the forms. To avoid this a piece of timber may be set with bolts along the face of the course while still wet, and as the concrete sets it grips the bolts and will hold the timber in place and protect the edges while the forms are being moved. When ready to continue the work, the nuts can be removed and the timber taken out. Another difficulty is that the fresh concrete is very apt to spring the bottom plank away from the face of the set concrete, leaving a projecting and unsightly offset in the masonry. This can only be prevented by holding the plank rigid.

The coping is often finished by spreading over the top of the concrete about 1 inch of mortar, of the same character as that used in the concrete. This mortar must be put on as soon as the mass below has been placed, so as to secure thorough bond, and should be lightly chopped or spaded in. A more satisfactory method, from the point of view of lasting results, is to build the concrete an inch or two above the coping level, and then pick out or drive down any projecting or visible stone, thus leaving only mortar

on the surface. The mass should then be left an hour or two to "sweat" or discharge the surplus water, which otherwise will bring up laitance and leave a surface which will scale off later. The surplus mortar is then removed and the surface is rubbed with a wooden float, and the mass allowed to set after being covered. This method will give a durable and fairly smooth finish. If a rougher finish is desired the surface can be lightly brushed over with a whisk broom just after the initial set has begun. Many engineers, however, prefer the smooth troweled finish. To obtain it, the work should be put in the hands of an experienced sidewalk mason, or if such is not available, an ordinarily intelligent workman acting under instructions can produce fair results. The mortar is spread as just described, and before it attains the final set it is carefully troweled over with a wooden float, causing part of the cement next the top to rise to the surface and giving a smooth finish. (See p. 429 for edge finishing.) Division lines, if such are used, are cut with a special tool, something like a plasterer's float, but having a curved V-shaped iron on its under side, with which the lines are struck. If the workman is inexperienced, a few sample blocks should be made and finished off before he is allowed to attempt the main walls, and the troweling should be moderate in amount, as too much of it will draw the cement out of the sand and leave a weak stratum underneath. If the surface has to be patched, the top should be removed down to sound concrete, and the latter should be brushed with a solution of water with 10 to 20 per cent of hydrochloric acid, removed later with clean water. The mortar should be mixed and be allowed to stand until it has obtained an initial set, when it should be reworked and then put in place and be troweled over. This "retempering" of the mortar appears to remove the tendency to separate later from the old concrete, and patches 20 feet square have been placed by this method and have successfully stood both time and stress of weather. All freshly-placed coping should be protected in hot weather for a few days with canvas on plank or with moist sand.

The top course on which the mortar is laid should be not less than 3 or 4 feet thick, as shallow top courses are apt to open along the horizontal seam under stress of weather, and the effect is very unsightly.

Concreting During Frost.—As the temperature falls the setting of the concrete becomes more slow, and at near freezing-point the chemical action appears to be almost suspended, unless special protection is used or the materials are heated. For the latter process steam pokers are sometimes placed in the stone and sand piles, and the water is heated.* A still simpler method, and a very satisfactory one, is to turn exhaust steam into the mixer as it revolves. Another method is to cover with canvas the section to be built, steam coils or stoves being used inside, and the materials being heated as well. Where these or similar precautions are used concrete can be laid at zero

* Where heating is impracticable, setting can be accelerated ten to fifteen times the normal rate by adding to the water 3 to 4 per cent of a saturated solution of ordinary lime.

weather or lower, and if the mixture can be put in the forms at a temperature of 60° to 70° or over, it will rarely suffer any harm. Once the initial set has been well attained, the cold does not seem able to disintegrate the particles of the mortar, as it does when the mixture is frozen before setting, and on the return of warm weather the mass will become hard, although as long as the temperature remains low it will stay soft. Moreover as the concrete sets it generates a large amount of heat, and the thickness of the lagging assists materially in keeping this in, and so advancing the setting. The surface of the mass must of course be thoroughly protected from the frost with layers of plank, manure, soil, or other satisfactory covering until set; the sides are usually sufficiently protected by the non-conducting lagging. Lagging only 1 inch in thickness, however, affords insufficient protection after the temperature falls somewhat below freezing. Thus the form for one of the piers of a bridge dam on the Mohawk River, N. Y. was composed of 2-inch plank for the sides and of 1-inch plank for the rounded nose. During concreting the temperature suddenly fell to near zero, and on examining the concrete during the following spring it was found that the sides were unharmed while the nose had been frozen for a depth of several inches.

Points to be Watched.—The following are the chief points requiring attention in order to obtain good concrete work.

1. Have the forms thoroughly braced, and the lagging smooth and even inside.

2. Have the concrete wet enough and with mortar enough so that shoveling will make it into a dense mass, but not so wet that water lies on the surface, or the laitance will give trouble as described on p. 442.

3. See that the face is carefully spaded or forked, so that no stone will lie against the lagging.

4. When placing the coping keep its front and back edges to an exact line in the vertical plane or the coping edge will look wavy, as you look along it, and always be unsightly.

5. Put on the mortar coping as soon as the concrete is placed, or, better still, build the concrete a little high and drive down the aggregate so as to leave only mortar on the surface, as described on p. 446.

6. Do as little patching of the face as possible after the forms are taken down, and if patches have to be put on, undercut the edges or dowel the mortar so that it cannot fall out later.

7. Rub off the faces as soon as the forms are removed if a smooth surface is desired, using plenty of water, or the grinding will leave a mortar skin, and be sure that no form fastenings, as the ends of wires or tie-rods, appear on or are left close to the face, or in time they will cause unsightly iron-stains.

Time Occupied in Lockages.—The following information gives the average time in minutes occupied in maneuvers at different locks.

	A	*B*	*C*	*D*	*E*	*F*	*G*
Entry of boats..............	10 to 20 (for tows)	1 (for singles)	4⅓	2	1 to 10	....	3½ to 6½
Closing gates................	2	2	⅔	2	2	1½	1⅙
Filling or emptying...........	10	5	3⅓	5	6 to 10	7 to 10	6⅔
Opening gates................	2	2	⅔	2	2	1½	1⅙
Exit of boats................	6 to 10	1	4⅓	2	1 to 5	....	2 to 5
Total minutes.........	30 to 44	11	13⅔	13	12 to 29	11 to 46	14½ to 20½

A refers to the large lock at Suresnes just below Paris and *B* to the adjoining small lock. The former is 55 feet 9 inches wide in the chamber, by 526 feet available length; the latter is 39 feet 4 inches wide by 164 feet available length. The lift is 10.7 feet and the channel depth 10½ feet. *C* refers to a lock on the Canal du Centre in France, and *D* to lock No. 7 on the Kentucky River, Ky., which is 52 feet wide in the chamber by 148 feet available length, with a lift of 15½ feet and a least depth on sills of 6.7 feet. *E* refers to an Ohio River lock, 110 feet wide by 600 feet long, with a lift of about 8 feet. The shortest time given is for a single steamboat and the longest is for tows. *F* refers to the Poe lock at St. Mary's Falls, Michigan, 100 feet wide by 800 feet long, with depth on sills of about 21 feet and a normal lift of about 18 feet. The minimum time of 11 minutes was made with a single steamboat 350 feet long, and the 46 minutes cover the time needed from commencement to commencement of lockages of the full capacity of the chamber. *G* refers to lock No. 2 of the Soulanges Canal in Canada, with a width of 45 feet, an available length of about 260 feet, a lift of 23 feet, and accommodating boats up to 14 feet draft. The shorter time refers to a towboat and barge, and the longer time to a barge without power.

Siphon Locks.—A peculiar type of lock, originated in Germany by Professor Hotopp, is known as the siphon lock, because the valves which ordinarily control the filling and emptying are replaced by siphons. The operation is as follows (see Fig. 170).

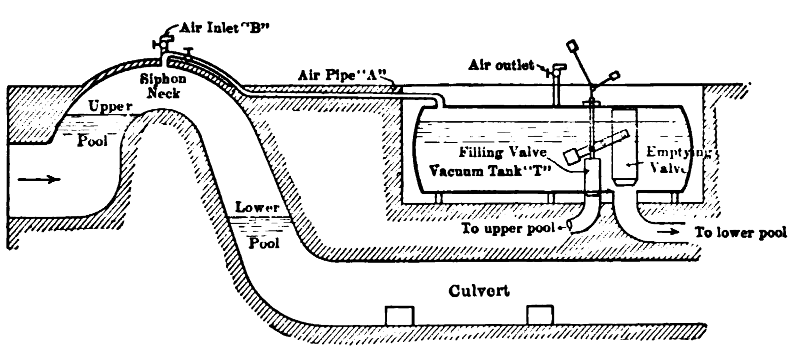


FIG. 170.—Section of a Siphon Culvert, showing Vacuum Tank and Connections.

The tank *T* is an air-tight receptacle connected by separate pipes with the upper and lower pools, and by other pipes with the siphon necks which form a part of the culverts.

This tank is first filled with water from the upper pool. The connection is then shut off and a connection is made with the lower pool by opening the other pipe. The weight of water in the tank is thus left unsupported, as it were, and so tends to cause a vacuum in the tank. To fill the lock, the air-pipe *A* from the siphon to the tank is opened, the air in the siphon is sucked into the tank, and the water rises in the former until it runs over the crest and on into the chamber. When the siphon is running full the air in the tank is sucked back and carried out, filling the tank again automatically, so that the vacuum is self-maintaining. To stop the flow *A* is shut off and another pipe *B* is opened which connects directly with the free air which flows into the neck and destroys the vacuum there, so that the water falls back to its natural levels. The emptying culvert is operated in a similar manner.

The advantage of this type of culvert is that it dispenses with the moving parts inseparable from ordinary valves, and has no parts below water which can deteriorate or become clogged or damaged. The first one was built at Krummesse, on the Elbe-Trave Canal in North Germany, and was completed about 1896. Its chamber dimensions were made $262\frac{1}{2}$ feet in available length, 56 feet in width, and with $8\frac{1}{4}$ feet least depth on miter sills. The width at the gates is $39\frac{1}{3}$ feet and the lift is about 9 feet. A single vacuum tank is used with a capacity of 918 cubic feet; the maximum volume of air to be drawn from one siphon is 388 cubic feet. Each culvert has a cross-section of 28.8 square feet, reduced to 18.7 square feet at the neck of the siphon. The pipe connecting the tank and with upper pool is 11.8 inches in diameter, and with the lower pool, 19.7 inches. The air-pipe connecting the tank and the siphon is from 4 to 6 inches in diameter. The lock requires about seven minutes to fill or empty. A tumble gate worked by compressed air is used for the upper lock gate; the lower gates are of the mitering type, operated by spars, cables and counterweights, also set in motion by compressed air. These arrangements were adopted in order to simplify all mechanical parts and to secure the operation of the lock through a series of valves, thus requiring the services of only a single lock-tender. The compressed air is supplied by the hydro-pneumatic method, by which a stream of falling water sucks down the air and delivers it into a closed receptacle and keeps it under pressure.*

As the operation of this lock proved satisfactory, the other locks in the canal were built on the same principle. There are thus seven siphon locks on the canal with normal lifts from 5.6 feet to 13.3 feet. The one communicating with the Elbe at Lauenburg has a low-water lift of about 18 feet. More recent examples are to be found on the Teltow Canal at Klein-Machnow, near Berlin, completed in 1905; on the Oder-Spree Canal, and elsewhere. In America a lock of this type was built in 1908–1909 on the Oswego River, in a lateral canal at Oswego, N. Y., with a lift of about 11 feet. The chamber is 45 feet wide with 300 to 310 feet available length and a least depth of 12 feet

* A detailed description of the lock, etc., will be found in Engineering (London), May 4, 1900, and in the Proceedings Inst. C. E., vol. lii, 1903.

on the sills. In this lock a separate vacuum tank is used for each wall, and the lock gates are of the mitering type, operated by spars and electric motors.* The culvert of the north lock wall of the lock at Little Falls, N. Y. (Fig. 160, p. 406), was built on the siphon principle, in order to save the large additional amounts of rock excavation and concrete which a low-level culvert would have required.

The siphon lock is not desirable for rivers unless in exceptional cases, as the greater the flood variation the greater must be the size of the vacuum tanks, since the neck of the siphon must always be at an elevation which will prevent the water from running over the crest of its own accord. As regards the limit of lift, we have been informed by some of the German engineers who have administered these locks that in their opinion the system could be applied to differences between upper and lower water levels as high as 24 or 25 feet, although others recommended a limit of 20 feet. This type of lock has given very satisfactory results, and in the case of the Oswego lock (allowing for the cost of the electric operating machinery which would have been used with culvert valves) was cheaper in first cost than a design made for the same structure with culvert valves, besides dispensing with their usual troubles of operation and repair.

In the German locks the siphon portions of the culverts from water line to water line, as well as the vacuum tanks, were made of iron; in the Oswego lock they were formed in the concrete masonry with no special lining, the necks of the siphons projecting above the coping being made of reinforced concrete.

Cost of Construction and Maintenance.—The cost of a lock, including cofferdam, excavation, construction of walls, backfilling, valves, gates, and all labor and material needed to make it ready for operation usually varies from $15 to $20 per cubic yard for masonry of cut stone, and from $14 or $18 per cubic yard for masonry of concrete. If pile foundations are used the cost will be increased from $2 to $3 per yard. These figures are obtained from examples of locks built within recent years, the wide variation resulting from differences in cost of material, accessibility of site, and other causes incident to all engineering work.

The cost of maintenance and operation depends largely on location and traffic. In rare cases one employee can attend to the lock, but usually not less than two are required, and where traffic is heavy three shifts of operators may have to be employed. For a lock on a river of ordinary size with moderate traffic and with a movable dam, from four to six men are usually sufficient. Maintenance charges (not including wages) for such a structure may average from $1500 to $2500 a year; for the lock alone perhaps $1000 less. The cost of operation and maintenance for nine years of 31 locks and several fixed dams on the Fox River, Wisconsin, averaged about $1800 a year for each location; these locks had only one lock-tender, at a low rate of pay. The similar cost for five years of ten locks with two fixed and eight movable wicket dams on the Kanawha River, W. Va., was about $4000 a year. These locks and dams had from two to five or six

* See Engineering News, Nov. 17, 1910, for a description of the design and operation of this lock.

employees, and the cost given does not include cost of renewal of the movable portions of the dam.*

Data collected by one of the Panama Canal Commissions and by another U. S. Commission gave the average cost of operation and maintenance of a waterway system as from 1 to 1½ per cent of its original cost.

* Annual Reports, Chief of Engineers, U. S. A.

CHAPTER III.

LOCK GATES AND VALVES.

GATES.

Miter-gates. (Fig. 171.)—The gates used for closing each end of a lock usually consist of a pair of symmetrical leaves, movable about a vertical axis, shutting against each other at one end and against the miter-sills at the bottom, and abutting against the hollow quoins at the other end. That part fitting into the hollow quoin is called the heel, while the opposite end is called the toe, and in old construction, where a post formed each side of the gate-frame, they were called respectively the heel- or quoin-post and the toe-post. The gates rest on shoes which turn upon pivots or pintles at the bottom of

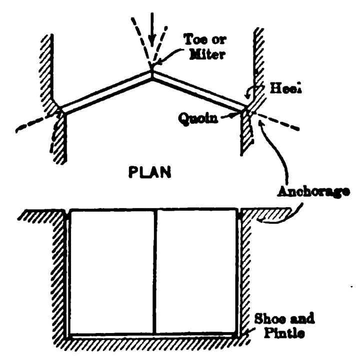


FIG. 171.

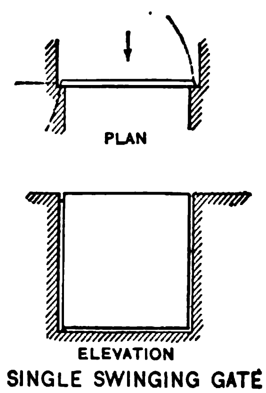


FIG. 172.

the heel portions, and they are held vertical by means of collars or anchor-bars fastened to the top of the masonry, and passing around pins in the bonnets or tops of the gate-heels. A roller has sometimes been placed on the bottom of the gate near the toe, traveling on the floor and relieving the strain on the anchors. The gates are usually operated by means of spars or chains, or both, connecting with the top of the gate near the toe and with suitable capstans located on the walls.

The miter-gate, after more than a century's use, has become the standard lock-gate, as it is simple and economical in building and operation, is reliable and easily repaired, or replaced, and gives less trouble from the effects of accidents than any other type.

Single Swinging Gates (Fig. 172) may be seen on many of the older locks of narrow width, but they are rarely applied to the locks used in river improvement. They are practically a single leaf of the miter-gate. Sometimes they have a balance-beam

extending back over the wall and serving as a lever for their movement. At the toe they are supported against a shoulder in the wall. This type is much in favor in France for narrow locks, as the gates are of simpler construction than the mitering type and require the services of a lock-tender on one wall only. They necessitate, however, longer walls for their recesses.

Lift Gates (Fig. 173) are in use on the Teltow Canal near Berlin and at the Little Falls lock on the Barge Canal of New York State. They resemble large counterweighted sluice gates, and are raised and lowered vertically, being suspended from towers on the lock walls. They have the merit of being accessible at all parts, and as far as used have been very satisfactory. They are more expensive than mitering gates and are of limited headroom, although they usually effect a saving of masonry.

Rolling Gates (See Fig. 174 and accompanying illustrations) are in use in Europe (as on the Bruges Ship Canal), on the Nile, on the Ohio River* and elsewhere. In the locks on the latter stream, where the cham-

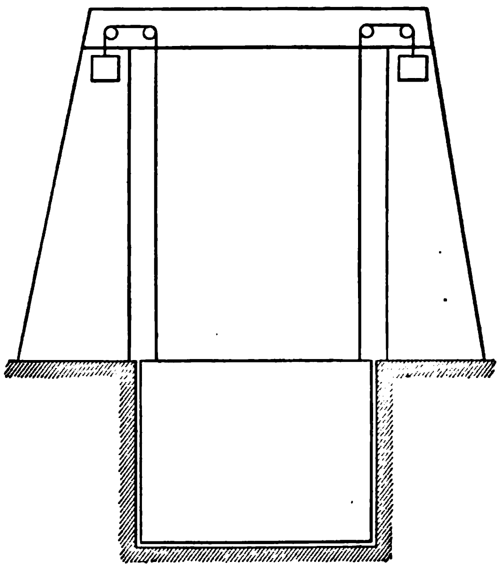


FIG. 173.

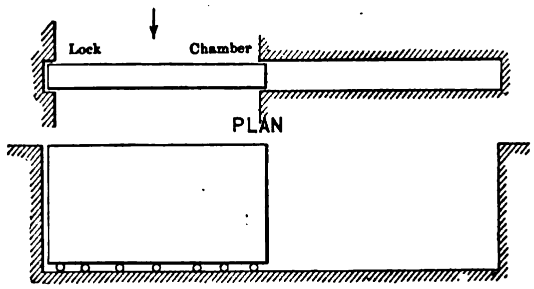


FIG. 174.

bers are 110 feet in width, the gates are supported on a heavy track, and present an appearance not unlike a long box-car. When not in use they are rolled back into recesses in the bank. There are, of course, two gates, one at each end of the lock, from about 16 to 20 feet in height, and about 14 feet in width, and with a length of 118 feet. They are built of steel, trussed like a bridge. The movements are made by steam-power with heavy chains wound upon drums, and considerable difficulty was encountered in those first built with the breaking of chains and wheels, and also with the breaking

* See Proceedings Am. Soc. C. E., 1908; Improvements of the Ohio River, Major W. L. Sibert, Corps of Engineers, U.S.A., by whose courtesy the accompanying cuts are reproduced. Forged steel wheels are now used for these gates. The tidal lock on the Charles River at Boston, Mass., with a chamber width of 45 feet and a maximum lift of 10 feet, is also provided with rolling gates. These are operated by electricity and heated internally by steam pipes, as the lock has to be operated all the year. The filling and emptying valves are placed in the gates, and are of the plain sliding lift type. Each gate has also a large flushing valve near the surface for passing off floating ice.

FIG. 1.—Partial View of Upper Gate, Showing the Operating Chain and Drum.

FIG. 2.—Gate during Construction.

FIG. 3.—View of Supporting Wheels and Base of Gate.

TYPICAL ROLLING GATES OF THE OHIO RIVER. CHAMBERS 110 FEET CLEAR WIDTH.

of the axles upon which the gates rest. The recess, 120 feet long and reaching back into the river-bank, was also expensive in construction, and débris was liable to accumulate in it at the time of locking. These difficulties have since been overcome, and in most of the later examples the gates have proved very satisfactory.

With another type, occasionally used, the gates are suspended from and roll upon an overhead girder which is swung up to a vertical position when the gates are in the recesses. Examples of these are to be found on the Nile at Assuan, and at a few places in Europe.

Tumble-gates (Fig. 175).—A gate with horizontal axis, known as a "tumble-gate," forms a type which has long been in use on the Erie Canal, N. Y., and in Europe. These gates consist of a single leaf hinged to the lock floor, the boats passing over it when it is lowered. They are maneuvered by chains passing over hand-winches on the walls. On the Elbe-Trave Canal in Germany hollow gates of this type are in use, and are raised by forcing air into the compartments (p. 449.)

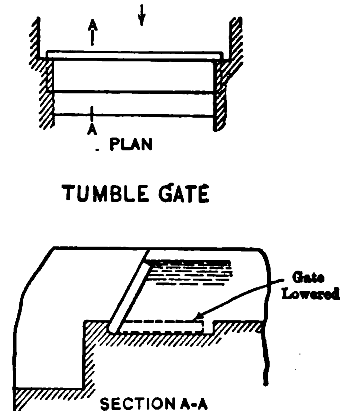


FIG. 175.

Many Russian engineers favor the use of the tumble-gate for locks of chamber widths exceeding 55 feet, intermediate supports or braces being provided when the gate is up.

Hydraulic Gates. (See cuts in Chapter IX.)—It has been suggested to apply the principles of bear-trap and drum dams to lock gates, in order to secure a gate which could be operated automatically, and several ingenious devices have been proposed for such an application. Gates of the former type were built in 1895 for use at the lock of the Sandy Lake Dam, in Minnesota, and were made 40 feet in length with a rise of 13 feet. They were used as sluice-gates also. An upper gate of the Chittenden drum type was built about 1901 at Lock No. 2, on the Mississippi River, near St. Paul.

Automatic gates which can be raised and lowered easily possess certain advantages over mitering gates, such as permitting the washing out of deposit from the chamber and entrances, etc., but they are more difficult to keep in working order, and more tedious and expensive to repair, owing to the number of parts, as journals, etc., always under water. Cases are rare where the simple reliable miter-gate will not be found preferable to any other.

Calculations for Lock Gates.—*General.*—In the following calculations only those types of gates will be examined which fall within the ordinary practice of the engineer. The rarer types, such as arched gates, have been discussed in other works.*

* See "Mitering Lock Gates," First Lieutenant Harry F. Hodges, Corps of Engineers, U. S. A., Government Printing Office, Washington, 1892. An analysis of the arched gates of the St. Mary's Falls Canal is published in the Annual Report of the Chief of Engineers, U. S. A., 1895, Appendix LL; and a full list of literature on Lock Gates will be found in the Report of the Deep Waterways Commission, Washington, 1900, pp. 200–207.

The simplest form of gate is that of beams placed squarely across the lock chamber, and supported against the masonry at each end. This style has been used on canal locks, as before described (Fig. 172). The strains in such gates are those of a simple beam, or, where the gate is large enough to require truss framing, they can be analyzed graphically or by moments, in the methods employed for framed structures supporting uniform loads.

The next form of gate, and the one which is practically universally used, consists of two leaves, and is known as the mitering type (Fig. 171)

Gates of this type are subject to the following strains:

(1) The pressure of the water.

(2) The reactions from the miter, the quoins, and the miter-sill.

(3) The weight of the leaf.

(4) The upward pressure from the water under the bottom beam. Shocks from craft, twisting from drift caught between the miters or behind the heel, will also induce strains, but they cannot be calculated. Experience has shown, however, that a gate designed to meet ordinary strains and properly connected, is very rarely disarranged by drift, and will stand in addition a considerable blow from a boat.

The pressure (4) seldom needs consideration, as it is usually offset by the weight of the framing, and where it is not, a proportion of the friction against the quoin may be taken into account. This upward pressure is a maximum when the gate has its maximum load, and at that time the friction is also a maximum. If it is not desired to make any allowance for the effect of the latter, weights must be added to the gates (as has sometimes to be done in practice) or other means taken to counterbalance the upward pressure.

The weight (3) in wooden gates is supported by wooden struts or by diagonal ties of iron, as will be mentioned later on. (See also cuts on pages 491 and 472.)

The strains from (1) and (2) are usually those which determine the proportions of the framing. The reaction from the miter-sill affects only vertically framed gates, except in so far as it supports the lowest beam of a horizontal framing.

In determining the head to be provided for in calculating gates for river locks one of the most important points to be considered is that in stationary dams there is usually a difference of level between the pools until, and sometimes after, the water has risen to the top of the walls. On one of the Kentucky River locks there is a difference of 9 feet between the pools when the lock is "drowned out," or flooded by the river. Such a condition requires the upper part of the gate to be made strong enough to stand the head produced by the difference between the pools at the varying stages of the water, a difference which, in locks already built, can generally be determined from gauge records. In new locks it can only be approximated from a study of general conditions, or by comparison with other locks similarly placed. On canal locks and those on rivers with movable dams these variations do not occur.

The lower gates, both with fixed and movable dams, should be proportioned so that they will withstand a full pool above, and a reduced pool below, since the latter condition will occur sooner or later through the leakage in the fixed dams caused by wear, and the possibility of repairs to the movable dams. In certain cases the upper gates may have to be used as a cofferdam, and it should be seen that they will stand the accompanying head of water without danger. This condition, however, is usually covered by providing for the ordinary pressures.

Water Pressure.—*Horizontal Framing.*—Let P (Fig. 176) be the pressure per lineal foot on any beam K of a horizontally framed lock gate AB, H the head of water to the center of K, and w the height of the beam in feet. If the beam supports sheathing, w

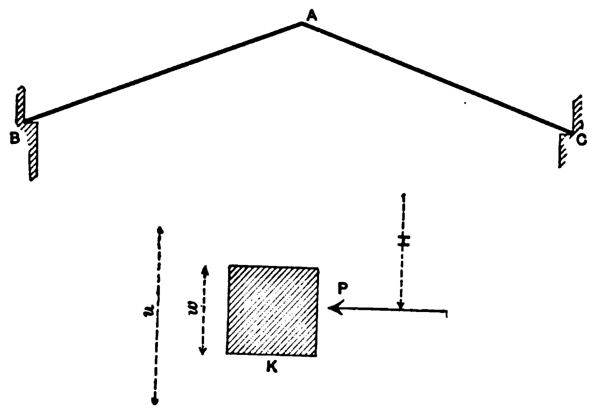


FIG. 176.

must of course include the total depth of water pressure supported by the beam, and if there be water below, H will be the net head.

Then P equals

$$w \times 1' \times H \times 62\tfrac{1}{2} \text{ lbs.} = wH \times 62\tfrac{1}{2} \text{ lbs.}$$

per lineal foot. The bending moment M, if there are no concentrated loads from valves, etc., will be

$$\frac{P \times (AB)^2}{8} \times 12 \text{ inch-pounds} = \frac{3wH \times 62\frac{1}{2} \text{ lbs.} \times (AB)^2}{2}.$$

The section required for this bending can then be found by the usual formulas $M = \frac{sI}{c}$ and $I = \frac{w_1 t^3}{12}$, where s = the extreme fiber stress per square inch, c = distance from neutral axis to extreme fiber, t = thickness of beam, w_1 its width, I = the moment of inertia of section, and M is as just stated.

Where K has to support concentrated loads from valves, we have a combination of distributed and concentrated loads, which may be calculated as follows:

Let CD (Fig. 177) be a beam supporting two valves which produce loads on CD at their points of support, each equal to Q. The beam has then to carry the valve loads, the proportion of water pressure on the lengths a and c between the valves, and the

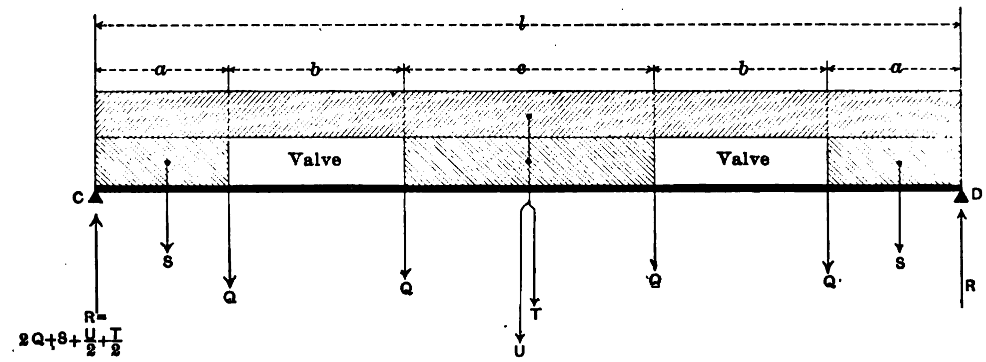


FIG. 177.

proportion of uniform load from the water pressure in the panel ECD above (Fig. 178). The proportion of the pressure on a between CD and F which is carried to CD is

$$a\times\left\{\frac{e(H+d)}{2}+e\cdot\frac{e}{2}\cdot\frac{1}{3}\right\}\times 62\tfrac{1}{2}\text{ lbs.}=\frac{ae}{6}\left\{3(H+d)+e\right\}\times 62\tfrac{1}{2}\text{ lbs.}=S.$$

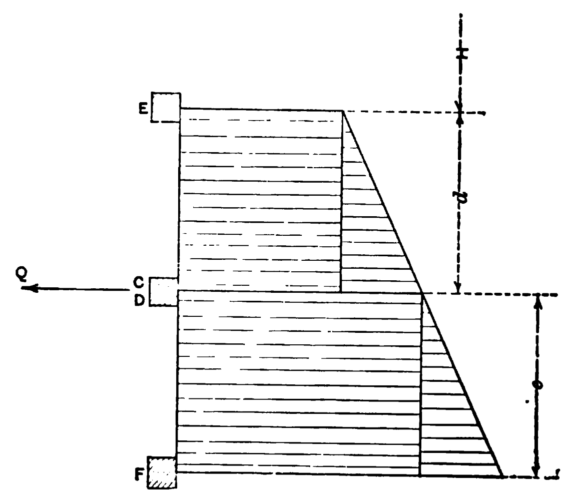


FIG. 178.

Similarly the proportion of pressure on c is

$$c\times\left\{\frac{e(H+d)}{2}+\frac{e^2}{6}\right\}\times 62\tfrac{1}{2}\text{ lbs.}=\frac{ce}{6}\left\{3(H+d)+e\right\}\times 62\tfrac{1}{2}\text{ lbs.}=T.$$

The proportion of the uniform load between E and CD, extending the length of the beam, which is carried to CD is

$$\left\{\frac{ldH}{2}+ld\cdot\frac{d}{2}\cdot\frac{2}{3}\right\}\times 62\tfrac{1}{2}\text{ lbs.}=ld\left\{\frac{H}{2}+\frac{d}{3}\right\}\times 62\tfrac{1}{2}\text{ lbs.}=U$$

The reaction at C is therefore $2Q+S+\frac{U}{2}+\frac{T}{2}=R.$

The bending moment at the center is then

$$\left\{R\cdot\frac{l}{2}-Q(b+c)-S\left(\frac{a+c}{2}+b\right)-\frac{U}{2}\cdot\frac{l}{4}-\frac{T}{2}\cdot\frac{c}{4}\right\}\times 12 \text{ inch-pounds.}$$

From this we can find as before the section required. As the center of the beam is the point of maximum moment, it should be seen that it is not too much reduced by notching for the gate-straps which are usually placed there for the purpose of construction.

Where more than two valves are used the process of analysis is similar to that given above.

In a metal gate where cover-plates are used it will of course be necessary to find the moment at one or two other points in order to determine the length of plate required.

The calculations for the upper beams of a gate are the same as used for beams with uniformly distributed loads, as given in the first example.

The foregoing calculations have not taken into account the possible support afforded to the beam through its being connected with those above and below, which in a wooden gate are often under-stressed as compared with the beams carrying the valve loads. Such support exists, but its amount is uncertain, as it depends on theoretical perfection of jointing which may or may not exist, and which in any case is liable to change as the gate wears. It is best therefore to proportion each beam so that it will carry its load independently.

It is frequently necessary in wooden gates where valves are used to reinforce the valve beams so they will carry the total load with safety. This is done by framing a metal girder between them, with diaphragm or vertical plates to which the valve journals are attached; the metal and the beam combined thus furnish the strength required.

The bottom beam of a gate which rests against the miter-sill can be considered as partly or wholly supported by it, and will therefore require no unusual section.

End Reactions.—In addition to the water pressure on its face, each beam is subject to end pressure from the opposite gate and from the hollow quoin. Thus if P' (Fig.

179) be the total water pressure on a beam inclined at an angle α, and T the reaction at the toe, we have, by moments about B,

$$P' \times \frac{AB}{2} = T \times AB \sin \alpha, \quad \text{or} \quad T = \frac{P'}{2 \sin \alpha}.$$

The reaction R at the quoin will be found to be of the same value as T, and inclined to AB at the same angle.

The component of T which acts along and produces compression in AB is equal to

$$C = T \cos \alpha = \frac{P'}{2 \sin \alpha} \times \cos \alpha = \frac{P'}{2} \cdot \cot \alpha.$$

This compression in wooden gates is usually assumed as distributed over the whole section of the beam, and produces a stress per square inch equal to $\frac{P'}{2} \cdot \cot \alpha \div a$, where a = the area of section in square inches. On the upper or compression side of the beam

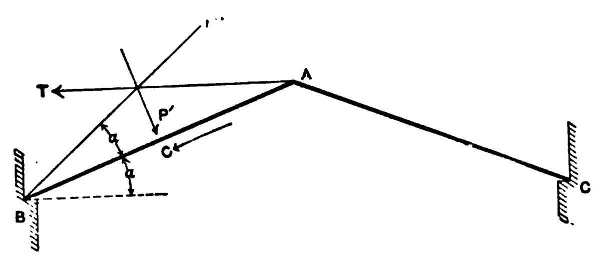


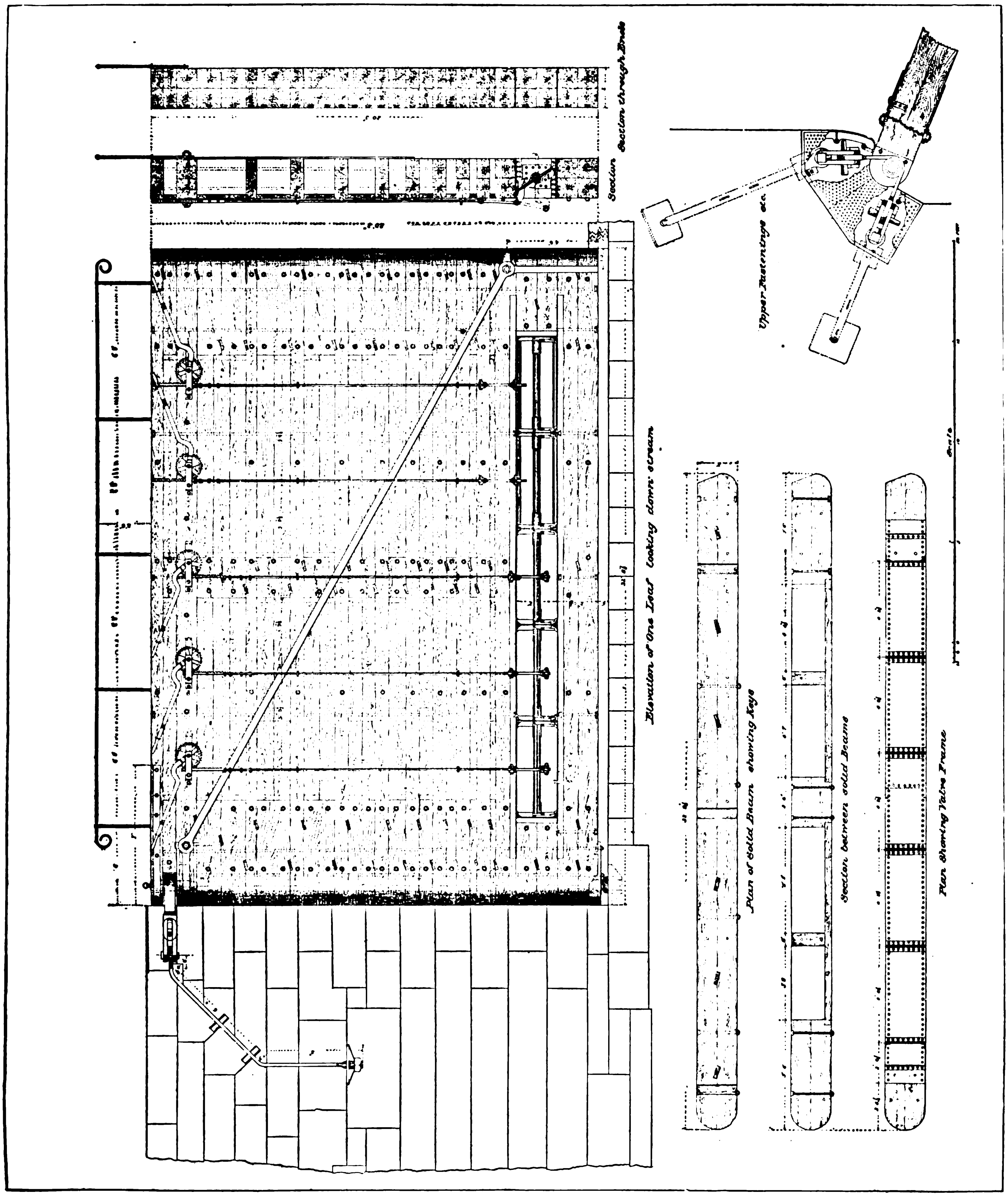
FIG. 179.

this stress increases that due to the water load; on the lower or tension side it decreases it, and this increase and decrease must be taken account of in determining the final section.* It will usually be found that the section required for bending will be sufficient to carry this additional compression without undue strain, since the point of maximum stress occurs only at the edges of the middle of the beam; the material near the axis of the beam thus receives very little strain from bending, and in all ordinary cases it will be found sufficient to carry the load from the end reactions as well.

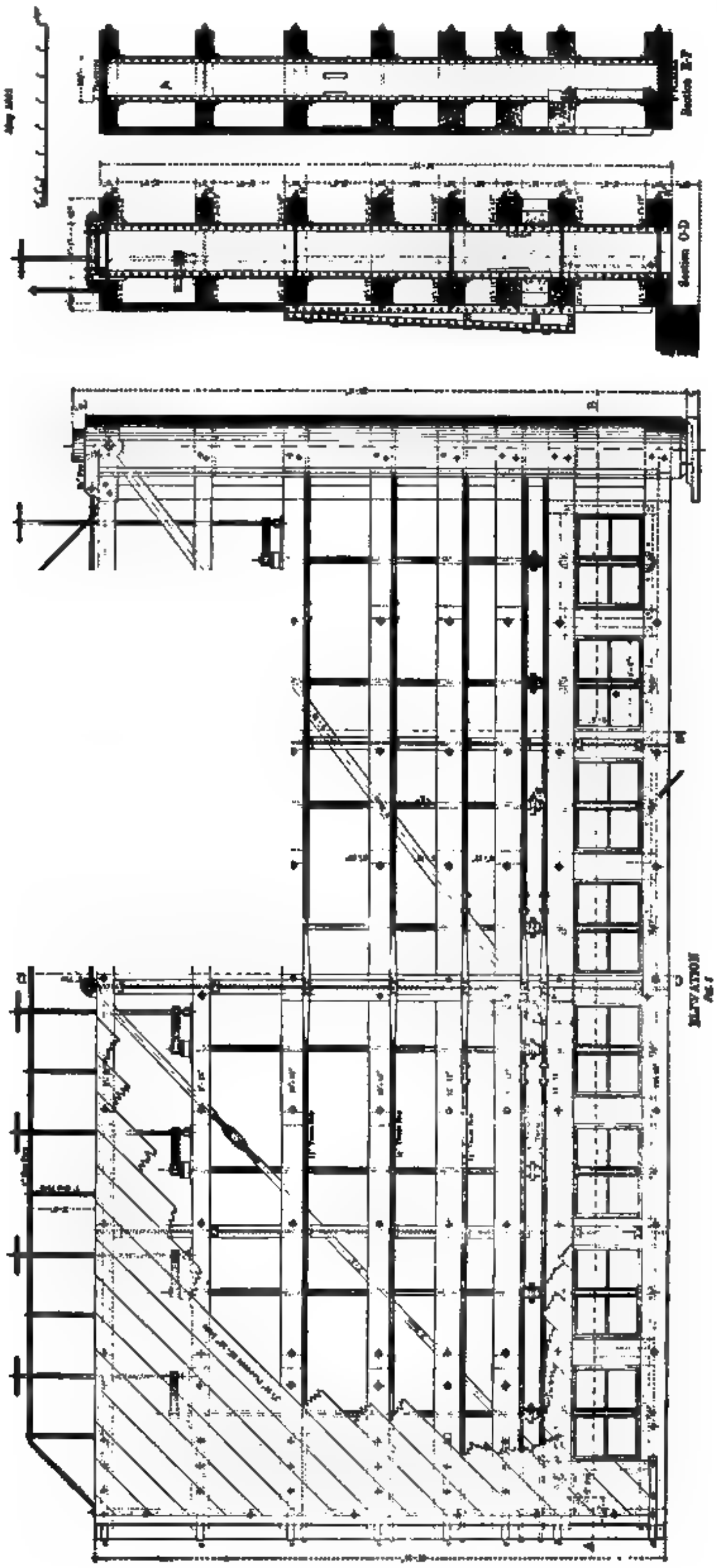
Vertical Framing.—Where the beams are placed vertically, as in a class of gates occasionally met with, they are supported at the bottom by the miter-sill and at the top by a horizontal girder. The strains in the latter are those from the concentrated loads of the verticals, while the strains in the beams themselves are found as follows:

Let H (Fig. 180) be the head of water, and d the distance from the top support to the surface, w the width of water supported by the beam, R_1 and R_2 the reactions as shown. Then the pressure $P = wH \cdot \frac{H}{2} \cdot 62\frac{1}{2}$ lbs. $= \frac{wH^2}{2} \times 62\frac{1}{2}$ lbs.

* See also "Metal Gates," p. 469, at bottom, and "Miter Joints," p. 480.



GENERAL DRAWING OF A LOCK GATE AS USED ON THE KANAWHA RIVER, WEST VIRGINIA.



GATE OF THE KAMPSVILLE LOCK, ILLINOIS RIVER, ILLINOIS.
(Trussed Bowstring Type.)

Taking moments about the base we find

$$R_1 = \frac{PH}{3(H+d)} = \frac{wH^3 \times 62\tfrac{1}{2}\ \text{lbs.}}{6(H+d)}.$$

The bending moment at any point distant x from the surface is

$$\left\{ R_1(d+x) - wx \cdot \frac{x}{2} \cdot 62\tfrac{1}{2} \cdot \frac{x}{3} \right\} \times 12 = w \times 62\tfrac{1}{2} \cdot \frac{H^3(d+x) - x^3(H+d)}{6(H+d)} \times 12 \text{ in.-lbs.}$$

The section required may be determined by the formulas given on page 457.

Strain on Anchor-bars, Diagonals, etc.—The anchor-bars should always be proportioned to carry the weight of the leaf when swinging in air, and in the case of small

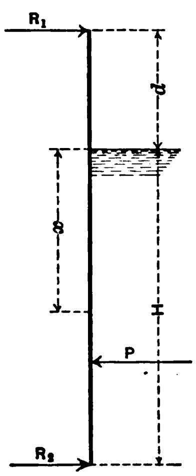


FIG. 180.

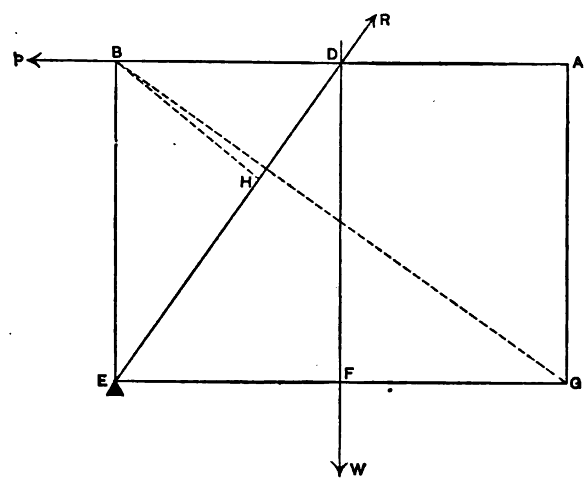


FIG. 181.

gates an increase should be provided in the section as a precaution against blows from craft, etc. The use of rollers to support the toe of the gate, with the object of relieving the pull on the anchors, has been almost entirely discarded, since any deposit over the track interferes with the wheels, and when they are a little worn their support becomes practically valueless.

If W (Fig. 181) represents the weight of a leaf swinging free, the pull on the anchor-bars, taking moments about E, is $P = \dfrac{W \times FE}{BE}$.

The thrust R on the pintle E, taking moments about B, is $R = \dfrac{W \times BD}{BH}$.

The strain on the diagonal BG is $\dfrac{W}{2} \times \dfrac{BG}{AG}$, since half of W is directly supported at E.

With a diagonal on each side of the gate the load on each will of course be one-half of this.

Unit Stresses.—In a wooden gate it will generally be found that only the tensile stress need be considered, as if the pieces are made strong enough in that respect the section will be ample for all stresses of compression and of shear. Those parts of a gate which normally sustain a full head, such as the beams just above the lower pool, may be designed for the minimum unit stresses given below, while those parts which are rarely subject to maximum loading, such as the beams of the lower gate below the lower pool which receive their maximum head only when the lower pool is drawn down, may be designed for unit stresses approaching the maximum.

Wood.—For gates of white pine a safe working tensile stress is 900 pounds per square inch, with a maximum of 1200 pounds. In a few cases the first unit stress has been made as high as 1200 pounds, but where the timber was subject to constant exposure the strains induced were found to be somewhat high. For oak and yellow pine a tensile stress of 1200 pounds per square inch may be used, with a maximum of 1600 pounds.

Steel.—The unit stresses used for steel gates have varied considerably. On the gates of the St. Mary's Falls Canal, Mich., (1895) 9000 pounds per square inch was used for the general framing, with a maximum of 10,000 pounds. One authority recommends 10,000 pounds per square inch for compression and 12,000 pounds per square inch for tension, while in several examples of gates for locks in rivers a maximum of 12,000 pounds per square inch has been used for compression and 16,000 pounds for tension, the beams or girders in these cases being assumed to receive no additional strength from the sheathing plates. On the gates for the proposed locks of the Deep Waterways from the American Lakes to the Atlantic (1900), the flange stress of the main girders was limited to 10,000 pounds per square inch, part of the sheathing being assumed to act as a flange-plate. The pressure between the wooden quoin-posts and the masonry was limited to 400 pounds per square inch.

As a gate is rarely subject to greater loading than that of static water pressure, it would appear safe to use moderately high unit stresses, say 11,000 pounds per square inch for compression and 14,000 pounds for tension, thus making a reasonable allowance for deterioration by rust.

WOODEN GATES

Vertical and Horizontal Framing.—Vertically framed wooden gates are practically obsolete in America, and for all river locks of ordinary size the simplest and best method of construction consists of horizontal beams extending in one length from the toe to the heel, well bolted and strapped together, their ends being shaped to fit the miter and the hollow quoin respectively. This gives a solid timber from end to end, and avoids the weakness of beams jointed into vertical heel- and toe-posts, such as are found in the older styles of gates.

Spacing of Beams.—In wooden gates, except where they are very small, the upper portion of the gate is usually paneled, that is, the beams are placed at varying distances, and the spaces between are closed with 2-inch or 3-inch plank. This is an excellent method of construction, as it saves material and weight, and also reduces the buoyancy of the gate in high water. The beams are usually made of the same width and depth as those of the lower part of the gate, and are placed at distances apart which will make each one support the same unit stress. (See Pl. 47 and cuts on pp. 461 and 462.) On the St. Lawrence River Canals in Canada with chambers 45 feet wide, wooden gates are used without panels, solid timbers being employed from bottom to top. The engineers prefer this type as it withstands accidents better than the paneled type.

Sizes of Beams.—The following are the horizontal widths of beams used in actual practice in certain horizontally framed wooden gates:

For chambers 27 feet wide, and normal heads up to 10 feet, the beams are 12 inches wide.

For chambers 36 feet wide, and normal heads up to 16 feet, the beams are 15 inches wide.

For chambers 55 feet wide, and normal heads up to 16 feet, the beams are 18 inches wide.

For chambers 45 feet wide and normal heads of $23\frac{1}{2}$ feet, with an extreme of more than 36 feet when the pools below are drained, the beams are 24 inches wide.

All the foregoing beams are of oak or long-leaf yellow pine.

In some cases two beams, placed side by side and keyed together, have been used instead of one single large beam; thus we find examples where two beams 9 inches wide, and two beams 12 inches wide, have been used instead of a single beam 18 inches and 24 inches wide respectively. This practice, however, is not a satisfactory one, as it takes more labor in construction and depends for proper strength on the exact fitting and duration of the keys, which matters are only realizable in theory. The cost of two small timbers is of course less than that of one large one, but the additional labor required in framing and handling the former will more than offset this. Experience with double- and single-timbered gates on the same river has shown, moreover, that the former are more easily injured by shocks and accidents.

Kind of Timber.—In most of the earlier gates white oak was used, and where sound sticks can be obtained, especially if they have grown on bottom lands, it is an excellent material and fairly durable. Yellow pine has been largely used of late years, and has proved very suitable. White and Norway pine are also good, but they are not often used except on Northern streams on account of the expense. Hemlock is not suitable above the water line, as it rots easily and is coarse-grained. Douglas fir has the advantage that exceptionally large and long pieces can be obtained, but it is easily shattered, and absorbs much water, soon becoming soft.

On the Manchester Ship Canal, in England, Demerara greenheart was used for all the gates in preference to metal on account of the satisfaction it had given elsewhere. This is an unusually durable timber, although it may be questioned whether this fact would compensate for some of the advantages to be gained in large gates by the use of steel. It is said not to be subject to attack by the teredo worm except in tropical waters, and hence is sometimes used for gates in sea-water.

Assembling and Erection.—Where gates are built of horizontal beams of full length these are held together by $1\frac{1}{4}$ or $1\frac{1}{2}$-inch bolts, running vertically through their centers, from the top beam to the bottom one, or by metal straps placed on the outside, and let in flush with the faces of the gate. Where the latter are used those at the heel and toe should be provided with turnbuckles for use in cramping the timbers together during construction. In each case diagonal straps must be used to hold up the gate; these are preferably formed of eyebars provided with turnbuckles, one end passing over a pin in the bonnet, the other over a similar pin near the lower end of the toe. (See Pl. 47.)

If the timber is to be dressed at the site, the pieces should be ordered $\frac{1}{4}$ inch larger than the finished size, to allow for planing. With oak or other timber liable to warp it is best to allow more than this. Sometimes the pieces are planed to exact size at the mill, but this plan has its drawbacks, as the timber usually shrinks and becomes more or less scarred in transit and in handling, and if it gets in wind there is no spare material for trimming it off. By using it ready planed some expense is saved, but the work is rarely as satisfactory as can be obtained by dressing at the site.

The beams should be finished so that when the gates are closed there will be no space between their ends, or between them and the miter-sills. If they are too long they will always leak, as the ends do not appreciably wear in use, and if too short, recourse must be had to filler-blocks, which are always liable to be knocked off. When the gates are built up in place it is best to leave the toes rough, and to saw them off to the exact length and bevel when the gates are finally swung, taking the measurement along each sill. These measurements should always be recorded on the drawings, as they usually vary slightly, so that when the gates have to be replaced the new ones can be cut to an exact fit.

Replacing Gates.—In new locks the gates are usually framed on shore and then built up in place piece by piece, but in replacing gates in old locks they should be set up complete, so as to delay navigation as little as possible. This can be effected by building them on shore on ways, and launching them when finished into the river. They are then supported under a barge and taken into the chamber, when they are hoisted in place by winches on the wall or by other means. A more rapid method, where the apparatus is available and the gate not too heavy, is to rig up one or two pairs of shears on the side of a barge, and to lift the gate off the ways with them. The barge, with the gate hanging from the shears, is then taken into the lock and the gate is

lowered into place. Where the gate is heavy, the opposite side of the barge has usually to be weighted with stone or earth, so as to equalize the loading. This method is used on the Welland Canal in Canada, where the gates close a chamber 45 feet wide, and are generally about 30 feet in height, built of solid timbers.

METAL GATES *

General.—In the last few years gates of metal have been coming into favor in the United States, and many examples of this class of construction are now to be found, varying from gates for chambers 36 feet wide as on the Green River, Ky., to those 110 feet wide as on the Ohio River. They will doubtless be more widely applied, since timber suitable for gates is becoming more expensive every year, and the quality of that which can be obtained is steadily deteriorating. From a comparison of several cases in 1906, metal gates cost at that time from 30 to 60 per cent more than wooden gates, depending on location and on size, the larger gates being relatively the cheaper. As to duration, the consensus of opinion seems to be that steel gates will last about twice as long as wooden ones. It should be remembered, however, that where the metal is constantly submerged it can never be properly cleaned and painted. The effect of water on steel, which is the principal metal from which plates and shapes are rolled in the United States, appears much more destructive than upon cast or wrought iron, and a few years' submersion will corrode and pit the metal to a very serious extent, rendering it a practical impossibility to secure a thorough cleaning. It might be possible to pump out the lock and thoroughly clean the gates every year, but this would involve a stoppage of navigation, and even on rivers of small traffic the loss and expense would scarcely justify the gain. In addition to this there would always be some recesses or portions of the construction, as for instance the under side of the bottom beam, which could only be cleaned and repainted with great difficulty, if at all.† For these reasons the parts constantly under water should possess a considerable excess of strength, and all the metal in them should be at least $\frac{3}{8}$ inch in thickness. The lower part of a wooden gate will usually outlast two upper parts, but of steel gates it is probable that the reverse will be found to be true.

* See Pls. 48 and 49.

† One of the most effective coatings for preserving ironwork under water is the ordinary red-lead and oil paint, while pitch has also given excellent results in some localities. In certain of the French seaport locks the metal is galvanized, which process is said to add about 10 per cent to the cost. At the Ymuiden locks of the Amsterdam Ship Canal duplicate sets of gates are kept on hand. About once every six years the gates are exchanged, and those removed are thoroughly cleaned and coated with coal-tar. This method also provides reserve gates in case of accident to the ones in use. The lock chamber is 82 feet wide and the largest gate is about 50 feet high. On the Monongahela River in Pennsylvania nickel steel has been used for gates and valves, but it has not withstood deterioration much better than ordinary steel. American ingot iron cost, in 1912, about 30 per cent more than steel, but it is believed by many engineers that it has resisting properties which render it in the long run more economical for use for river structures, barges, etc.

Adulteration of coating materials is not infrequent, and it is unsafe to use them without tests.

On the continent of Europe steel has replaced wood almost entirely, even in very small gates, but in England wood is still largely employed.

The objection is often brought against metal gates that they are more difficult to repair and more liable to injury than wooden gates. This, however, has not proved to be the case, and the metal gate has been found as well able to stand mishaps as the wooden one. Thus, on the Marne, where iron gates have been in use for over 50 years, it was feared that boats might pierce the sheathing and bend the framing, but experience showed that no more trouble occurred than with the older wooden gates. The steel arch-gates of the 100 foot lock of the St. Mary's Falls Canal in Michigan, less able to withstand blows than the girder or beam type, have been frequently rammed by boats, and have stood with comparatively little damage several very serious accidents.

Vertical and Horizontal Framing.—Metal gates for locks of ordinary size may be divided into two classes, those with vertical framing, and those with horizontal framing. In the former class the beams supporting the water stand vertically, and are supported by the miter-sill at the bottom and by a horizontal girder at the top (Pl. 48). In the latter class the beams are horizontal, as in a wooden gate, and are framed into vertical heel, and toe-posts, which in turn transmit the strains to the walls and to the opposite leaf. (Pl. 49.)

There is also a composite type in which vertical girders are employed to assist in carrying the loads from the plating to the horizontals, the latter forming the main supports. With this construction both horizontals and verticals act in resisting the pressure, and the precise loading of each can only be approximated by a tedious mathematical investigation. For this reason many designers assume that all the pressure is resisted by the horizontals, and make each vertical strong enough merely to carry the pressure to them from its own panel. This type is also more complicated in construction than a plain system of horizontal or of vertical framing, and as a rule is therefore not economical.

Where a gate is very shallow in proportion to its length, economy of metal favors the use of vertically framed gates. This type, however, has one or two disadvantages, namely, that the top horizontal girder brings a heavy strain high up in the wall, and that the recess in the coping necessary for it when the gate is open reduces the width required by the lock-tenders for the proper operation of lock, or else necessitates a wider wall in order to avoid such a reduction. Moreover, when the lower part of a vertically framed gate is rusted away the entire leaf must be renewed. With horizontal framing the gate could be cut in two at the beam nearest the water, and a new bottom riveted on, the upper portion being still retained in service.

Vertically framed gates are applicable to new locks only, on account of the girder recess above mentioned.

Design.—The economical design of a steel gate requires few members with moderately wide spacings, thus concentrating the strength and providing deep beams and a good thickness of metal throughout. The depth of beam is important, as a gate composed of shallow beams with long spans lacks stiffness, and this defect is apt to grow worse with use. For wide spacings, the use of buckle-plates is advisable. Although more expensive than flat plates, they are cheaper in the end, as they will carry a heavier load. They should be not less than $\frac{3}{8}$ inch thick at the bottom, diminishing to $\frac{1}{4}$ inch for the upper panels. All shop-work should be made as simple as possible, since curving or bending in order to save metal usually costs more than the material it saves, and in case of accident complicated pieces are more difficult to obtain and fit up.

The calculations for the beams are based on principles similar to those before given, but the analysis, for gates of any size, is usually more detailed and exact (see references in foot-note). The vertical sagging of the toe, apparent in all unbraced wooden gates, and often appearing in well-braced ones as they become old, is absent in metal gates, as the skin-plates hold the gate rigid in the vertical plane. Experience has shown, however, that diagonal suspenders or diagonal struts are necessary in order to give lateral stiffness, as metal gates are much more limber sideways than wooden ones, and if the suspenders or struts are omitted the gate will tend to warp. This tendency is increased if the beams are very far apart, and ample support will be needed to counteract it. Examples are found of narrow and high gates (as on some of the French canals) without straps or bracing, but the tendency of modern design is to use stiffening for all cases. The common practice abroad is to use an X-bracing, consisting of two struts or of one suspender and one strut, but in the United States it has been customary to use only one suspender. This is placed on the downstream side, the skin-plates being (usually) placed on the upstream side. The double or X-bracing, however, adds a greater stiffness to the gate. Some designers make the diagonal in two pieces connected by a turnbuckle near the center of the gate, but with no connection to any of the gate beams except at its ends. In several cases where the diagonal had been riveted to each of the beams, it became stretched or buckled between its points of support, due either to defective workmanship or to overstrain during maneuvers. Whatever method is used, the pieces should be able to withstand ordinary blows from boats as well as to carry the indeterminate strains of operation, and protecting fenders should be used where desirable.

The compression from the component C (Fig. 179, p. 460) requires a more economical treatment than with wooden beams. For straight-backed leaves the following formula may be used (Fig. 182):*

* "Mitering Lock Gates," p. 33; First Lieutenant Harry F. Hodges, Corps of Engineers, U.S.A. A later and more comprehensive discussion of the general principles of design as applied to steel lock gates will be found in Part I, Report of the Board of Engineers on Deep Waterways, 1900, and in "The Kinetic Theory of Engineering Structures," by David A. Molitor, 1911. It may be added that the gates should be designed for conditions in contact with the miter-sills and otherwise, since it is an impossibility in practice to secure perfect mitering.

Let C represent the maximum compression in the upstream flange, T the maximum tension in the downstream flange, D the depth of the beam, l its length, p the load per unit of length, α the angle of the sill, and λ and λ' the distance from the axis of compression to the up- and downstream sides respectively. Then taking all dimensions in inches,

$$C=\frac{1}{D}\left(\frac{pl^2}{8}+\frac{pl\cdot\lambda'\cot\alpha}{2}\right)\quad\text{and}\quad T=\frac{1}{D}\left(\frac{pl^2}{8}-\frac{pl\cdot\lambda\cot\alpha}{2}\right).$$

For the compression C_1 and the tension T_1 at any point distant x from the end we have

$$C_1=\frac{1}{D}\left(\frac{plx}{2}-\frac{px^2}{2}+\frac{pl\cdot\lambda'\cot\alpha}{2}\right)\quad\text{and}\quad T_1=\frac{1}{D}\left(\frac{plx}{2}-\frac{px^2}{2}-\frac{pl\cdot\lambda\cot\alpha}{2}\right).$$

The most economical shape for the beams for locks of moderate width and with ordinary lifts, consists of I beams, which can now be obtained in depths of 24 inches and under. They are cheaper than built-up girders, as they require less shop work, and where the section has to be reinforced, cover-plates can be added to the flanges. The beams or girders used should be of good depth, as this assists materially the stiffness of the gate. Holes should be provided in the webs for the drainage of rain- and flood-water.

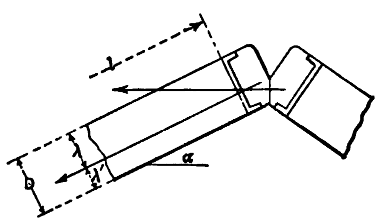


FIG. 182.

Where the gates support a considerable pressure it is sometimes necessary to plane off the ends of the lower beams so that they will have a close bearing against the heel- and toe-verticals, and thus transmit the compression from the other leaf. Stiffeners may also be used for this purpose.

The heel- and toe-posts, except with gates of extreme pressures, are usually provided with timbers bolted to them and shaped to fit the hollow quoins and the miter. This is an excellent practice, as the wood provides a tight and elastic cushion, and where the quoins are of masonry it will wear itself into fitting any irregularities, and where they are of cast iron it can be planed off to fit imperfect joints and alignment. (Pls. 48 and 49.) The wooden heel is objectionable, however, on the point of difficulty of renewal, and care must be taken in designing the gate to see that this is reduced to a minimum. Where this is done timber will often be found preferable to a metal heel cushion, owing to the difficulty of adjusting the latter to the unavoidable imperfections of construction. If these wooden cushions are used, they should be cut in two just below the pool level, and the portions above, which will rot first, can then be removed without disturbing the parts below water. This removal will be facilitated if the heads of the bolts are placed facing inwards along the axis of the gate, as more working room will be available for withdrawing the bolts.

It has been proposed where a metal cushion is employed to use strips of a soft metal, such as lead, set vertically in slots in the cast-iron pieces. The lead would then permit trimming to fit inequalities of the quoin.

The contact with the miter-sill may be formed by the bottom beam of the gate, or an additional wooden cushion may be used. The former method is the simpler, and therefore the preferable one for all ordinary cases.

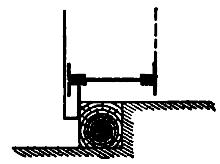


FIG. 183.

The upward pressure on the lowest beam, occurring when the gate stands closed under a head, can be reduced where necessary by placing the sheathing plates towards the downstream side, as shown on Pl. 48. Another method is to place the miter sill so that the gate closes over it, contact being obtained by projecting the sheathing as shown in Fig. 183.

The beams or girders above the center of the gate are usually made of the same strength as the middle one, so as to give resistance to blows from boats.

Arched Gate Formula.—For a gate of the pure arched type, that is, where the two leaves when closed form a segment of a circle, the formula used is $T = PR$, where T is the total compression in pounds produced in a vertical depth of 1 foot of the surface or ring of the gate or arch, P is the water pressure in pounds per square foot on one square foot of surface at the point in question, and R is the radius in feet of the surface supporting the water. T is of course the same all along the arch for the particular depth under consideration. This formula represents the water pressure only; the stiffness to be added for strains of operation, protection from boats, etc., is largely a matter of judgment.

Sheathing Plates.—These may be of buckled or of flat plates. The former are preferable, as they possess much more strength than the flat plates, and permit of wider panels between the beams without requiring vertical stiffeners. Some designers place the convex sides facing upstream, while others reverse this practice. In actual use one method appears to be as good as the other. The actual strength of the buckles is a matter of experiment, as it cannot be calculated. The following values were found for plates 3 feet square, well bolted down on all sides.*

Thickness.	Safe Load per Sq. Ft. (One-fourth Breaking Load.)
$\frac{1}{4}''$	1120 lbs.
$\frac{5}{16}''$	1540 "
$\frac{3}{8}''$	2240 "

Where the plates are supported on four sides, but not bolted or riveted down, the above loads are to be reduced one-half. If two sides only are supported the loads are to be reduced in the proportion of 8 to 5.

* "Pocket Companion," Carnegie Steel Co., 1900.

The resistance of flat plates under conditions such as exist when they are employed as skin plates is likewise a matter of uncertainty. The most recent and probably the most reliable experiments in this field are those of Professor Bach of Stuttgart, who gives the following formula for flat rectangular plates, subject to fluid pressure and supported on all sides:

$$t = cb\sqrt{\frac{p}{\left\{1+\left(\frac{b}{a}\right)^2\right\}f}},$$

in which t = the thickness of the plate in inches;

c = a constant = 0.61 for a plate riveted down all around;

a = length of plate between supports in inches;

b = breadth of plate between supports in inches;

p = intensity of fluid pressure in pounds per square inch;

f = maximum allowable tensile stress in the metal in pounds per square inch.

Plates thicker than ½ inch are rarely needed, even with high heads and panels of considerable size.

The spacing of the rivets in the plates, as regards securing water-tightness of the joints, need not be closer than 6 inches, and examples are in use where the spacings were made 9 inches, and have proved perfectly tight under heads of 14 to 16 feet.

Single and Double Sheathing.—The use of double sheathing for gates, that is, of plates on both upstream and downstream sides, has been confined chiefly to very wide locks. The object of the double plating is to provide water-tight compartments near the bottom of the leaf so as to secure buoyancy and consequent reduction of weight. This lessens the strain on the anchors, and facilitates maneuvers. In practice, however, they have hardly proved an unqualified success. It has been found difficult to keep the chambers free from leakage, and when the water gets in, the gate loses its buoyancy, and the second skin merely acts as an additional and useless load, defeating the very purpose for which it was put on. This style of gate, where used on rivers, has been found open to another objection, which is, that the upper or non-water-tight compartments gradually fill with mud. This can only be removed through the manholes, and with much trouble, and if left in it adds considerably to the weight of the gate.

It would appear that single-sheathed gates, although less rigid than those with double sheathing, are preferable for all cases which are likely to occur in the ordinary practice of the engineer. For the few cases which may require extraordinary dimensions, a study of the local conditions must of course decide the choice of type.

It may be added that both for single and for double sheathing the anchorages and maneuvering apparatus should be proportioned to take the heaviest strain that can result from the construction, that is, when the gate is swinging in air.

Combination Gates.—In order to avoid the loss of strength resulting from the rusting of iron or steel under the water, a style of gate has been proposed in which the parts constantly submerged would be of timber, while those above water would be of metal. We are not aware of any case where this idea has been put into practice, but there appears to be no reason why it could not be successfully applied. Many examples are met with abroad, however, of gates with metal frames and wooden sheathing, and the type has proved very satisfactory.

General Remarks on Design and Choice of Type.—(See also "Details of Construction, p. 476.") The supporting of the head of water is the chief function of a lock gate; its other function, and one of almost equal importance, is that it should be able to stand the impact of boats with a minimum of injury. This impact usually occurs in two ways. The first is the rubbing of the boat along the gate when passing in or out of the lock, but if the gate is provided with horizontal fenders harm will rarely result. The second, and fortunately a comparatively rare one, is where a boat strikes a gate when closed or partly closed. In this case the injury may vary from a wrenching of the framing to the breaking of the anchorage and the loss of the gate. If the gate breaks loose its fellow is usually carried away by the rush of water, and it often happens that the next pair below is lost as well.* As such an accident blocks navigation until the gates can be replaced (and usually they need extensive repairs in addition) it is evident that they should be designed as far as possible to resist such mischances. On rivers and canals with important traffic all gates should be made alike as far as practicable, so that by keeping one or two spare pairs on hand an accident at any lock can be promptly remedied. By varying the elevation of the miter-sills, and in some cases of the anchorages, it has been found practicable to obtain gates which would be interchangeable with locks of considerable difference in lifts. This practice was followed on the Mohawk River, N. Y., and at other locations on the New York State Barge Canal.

Among the points which experience has shown to be important in order to obtain a design adapted to meet these contingencies are the following. The gate, whether of wood or metal, should be of plain and simple design, free as far as practicable from difficult framing or bent pieces. It should, in other words, be built up of parts of which duplicates can be readily obtained and which can be easily repaired or replaced. The anchorage should be simple and accessible, and if it appears necessary to cover any portion of it with masonry, there should be left a suitable provision in the accessible portions, so that if a breakage occurs the buried iron work will not be injured, and the new anchor can be attached without having to dig out the masonry. The type shown on Pl. 50 is simple and accessible, and has a record of many years' successful use under all conditions of operation and accident. The hollow quoin should be of a type which will support the gate when struck from above, and not be a mere

* See Chapter X for details of such accidents.

surface to support a thrust at right angles to it, and the miter joint should be of ample width. The pintle should be designed so that the gate can fall over without wrenching or displacing it. This has been found of importance in replacing gates carried away by floods or other mischances, where no diver was available who could have reset the pintle if it had been displaced. Some engineers bolt the pintle to the masonry or set its base in a recess, while others prefer a "sliding" type which will permit of movement in case débris gets caught between the gate and the sill, or between the gate and the hollow quoin. (See "Pintles and Shoes," p. 477.)

The foregoing points are described more fully in this chapter under "Details of Construction." (See also Pl. 50.)

Choice of Type.—As regards a choice of the various types described at the beginning of this chapter (Figs. 171 to 175), the one which has stood best the test of economy in building and maintenance, and of general success in use with locks of all sizes, is the miter-gate. For small locks the single swinging gate is well adapted, especially where only one lock-tender is to be employed, but it requires longer walls than for mitering gates. The tumble-gate has also proved very satisfactory where used as an upper gate for canal locks for 4 to 6 feet draft, but it may be questioned whether it is adapted for general use, as in rivers silt and drift might prove troublesome, and there is the objection of the hinges being under water.

Rolling gates are desirable only in special cases, as on the Ohio River, since they are costly and the moving parts under water will cause difficulties in case of breakages or clogging. At the locks of the Assuan Dam on the Nile (31 feet 2 inches wide) rolling gates were used suspended from overhead bascule girders, but after their completion the engineers were not favorably impressed with the results.* Estimates made in 1906 for rolling gates as compared with miter-gates for 12 locks for a ship canal of very large dimensions were reported to show that the former would cost $5,000,000 more than the latter. In American practice the gates roll on a track under water; abroad they are usually suspended and roll on bascule girders.

Lift gates may prove well adapted to certain conditions, but experience with them is small (see p. 453), and they can only be used where the clearance under the gate when raised is to be limited. The towers and other accessories are complications from which the miter-gate is free. This type, however, usually effects a saving of masonry, as it requires a very short recess.

Hydraulic or automatic gates appear to have no advantage beyond that of theoretically simple operation. Where it is practicable to use turbines or other operating power, this advantage becomes comparatively small, and under any circumstances is of very questionable value in view of the complicated design and difficulty of repair in case of accident which this type involves.

* Proceedings, Inst. C. E., vol. clii., Part 2, p. 148.

The comparative features of wooden and metal gates have been described in the preceding paragraphs on "Wooden Gates" and "Metal Gates," pp. 464 and 467.

The relative advantages of arched and girder metal gates are set forth in the following paragraphs. The remarks apply equally to small and to large gates. (See also "Cost of Gates," New York State Barge Canal, further on in this chapter.)

Gates for Large Locks.—The question of the design of gates for locks of unusual size was taken up in 1899 by a Commission of Engineers, for the United States Government, in connection with the surveys for a proposed ship canal between the inland lakes and the Atlantic.* The widths of the proposed locks were 60 and 80 feet, and the lifts from 30 to 41 feet, with depths on the sills from 21 to 30 feet. This required some of the gates to be designed for possible heads of over 70 feet, far in excess of any yet employed. The investigations therefore were exhaustive, and the results are the more valuable as they combine international opinions and practice.

The most economical rise of sill was found to be about one-fifth of the chamber width, or about twenty-one degrees. The ordinary mitering type was adopted for the gates, as it possesses more than any other the merits of reliability, simplicity, and strength, and steel was selected for the material, wood being inapplicable to the conditions. The pure arch form, which has hitherto been principally used for wide-span locks, was discarded in favor of a girder section with a straight downstream face and an upstream face straight for about one-third of its length, and then curved in toward both ends. The middle was made 4 to 4½ feet wide, and the ends about 2 feet wide. Comparative estimates showed that an arch would not save more than 10 per cent of weight, while it would possess the disadvantages of being much more expensive in manufacture, and of having less stiffness and strength to withstand shocks, since its section would be thinner than that of the girder gate. It would also require a deeper recess in the walls. For these reasons the girder section was found to be more economical and suitable.†

All the plating was to be of single sheathing, flat plates from ⅜ to ½ inch thick being used. The framing throughout was of the horizontal type of equal spacing, only two vertical diaphragms being put in each leaf, and these were employed solely for the sake of stiffness. The quoin- and toe-posts were of wood, except for gates for locks 80 feet wide, with heads of over 30 feet, in which cases steel had to be used, as wood was unequal to the pressure.

* Report of the Board of Engineers on Deep Waterways between the Great Lakes and the Atlantic Tidewaters, Washington, D. C., 1900. Appendix on Lock Gates by Henry Goldmark and S. H. Woodard.

† The comparative ability of the arched and of the straight-girder type to withstand shocks may be illustrated by supposing a boat to strike, with sufficient impact to force the leaf open, the lower side of a girder supporting a head of water. In an arched girder, which depends for its equilibrium under a load on a fixed direction of the resultant pressures at the toe, the framing is suddenly submitted to intermediate bending stresses for which it was not designed, and which, if the pressure is great, may distort it seriously. In a straight girder, which acts as a beam, the point of support is merely changed from the toe to a point a few feet distant (as the point of impact when a boat strikes a gate is almost invariably near the miter), and the supporting power of the girder remains practically unimpaired. Gates of the straight mitering type were used for the Panama Canal.

In order to overcome the upward pressure under the gate, as was necessary in certain cases, the arrangement was adopted of using a narrow vertical plate for the bottom panel, stiffened with angles and attached to a main frame just above by cast brackets. When the gate was closing the main frame would pass over the sill and be stopped by shoulders on the brackets which bore against the sill, about as shown by Fig. 183, p. 471. These shoulders were to be about a foot wide, up and down-stream, the spaces between them being closed by a horizontal plate stiffened with angles. By this means the water would press upward only on the horizontal plate, instead of on the whole width of the gate. The sill was to be trimmed to fit this special construction.

DETAILS OF CONSTRUCTION, ETC.*

Anchor-bars.—The simplest style of anchorage consists or two ordinary eyebars provided with turnbuckles, one eye of each bar being placed over the gudgeon or bonnet pin, and the other over a large bolt cemented in the masonry, and provided with a collar or casting at the top to distribute the pressure. Sometimes the masonry ends of the bars are upset and threaded, and provided with nuts which bear against angles or castings, instead of using bolts. (See Pl. 50.) The bars are placed horizontally in recesses which are covered with cast-iron or reinforced concrete plates, and are thus accessible at all times for examination or adjustment. They require a minimum of forging, which is a desirable feature, not only because of economy, but also because that class of work is not always reliable. Not more than one bolt should be used to each bar, as where two or more are used it is not possible to set them to an exact enough position to secure an equal strain on each, and as the result only one of them actually bears the load. Usually only two anchor-bars are used for each gate, one placed in continuation of the line of the gate when closed, the other placed a little back from the same line when the gate is open, so as to avoid bringing the bolts too close to the edge of the masonry. This gives the gate a slight tendency to overbalance when open, which is usually counteracted by the second anchor-bar. Should the motion exist, however, it will be so slight as to be of no practical importance, and it can be stopped by placing a block against the end of the bar. Another type of anchor has been used for gates of moderate size (as on the Kanawha River) as well as for very large gates (as on the Panama Canal), and consists of separate bars attached to a framing or to other bars which are buried in the masonry. (See illustrations on pp. 461 and 462.) This type has the disadvantage of difficulty of repair in case of breakage or overstrain of the buried portions, or of damage from rusting. While it has been in satisfactory use for many years, it appears to have no special advantages for gates of ordinary size over the two-bar type just described, which is simpler and has

* See also preceding paragraphs in this and in the preceding chapter.

also successfully stood all conditions of operation, as well as accidents where the gates have been torn loose.

The hinge is formed by a steel pin (the use of bronze for pins is unnecessary, as the wear on them is very slight), and a cast thimble or washer should be provided to fit over it where the anchor-bars come, so that the wear will take place in the casting rather than in the bars. Access should be provided for oiling the pin. The recesses in which the anchor-bars are placed should have a slight bottom slope towards the chamber, so that rain and flood water will drain out. Cast or wrought plates are used to cover the recesses, fastened down so they will not be carried away by floods. In some cases reinforced concrete cover plates have been used with satisfaction. The strain on anchor-bars, except for very large gates, should not exceed 5000 pounds per square inch, as they occasionally have to withstand the impact of boats, etc.

Pintles and Shoes.—Two forms of pintle are in general use; in the first one a steel pin forms the support and is set into a cast bedplate, the latter being usually set into and flush with the masonry, and in the second, the support and the bedplate are cast in one piece and also set into the masonry. (See Pl. 50.) The removal of the pin of the former design, when worn, is usually very difficult, as the surfaces, even if originally greased, adhere so firmly that steel wedges have sometimes failed to separate them. The pin is also liable to quick rusting if made of ordinary steel, and in some cases it has not proved able to stand the hard wear. Sometimes a hard bronze pin is used or a hard bronze bushing is placed in the shoe. Pintles of the second design have proved equally and sometimes more durable in practice, and the design is in some respects more simple. In a third type, rarely met with except in old locks, the pintle and bedplate are separate, and designed so that the former can slide upon the latter. The object of this is that if débris gets between the heel-post and the quoin, or between the gate and the sill, the bottom of the gate can slide out and so prevent wrenching. A later modification of this has been to use a pintle and bedplate of the customary one-piece design, and to put a flat base plate beneath, so that the pintle will slide upon the lower plate. At Lock 41, at Louisville, on the Ohio River (1912), a sliding type was used, the pintle plate being made 3 feet 8 inches in diameter and the base plate 3 feet 10 inches, allowing a total movement of 2 inches. The pintle itself was of nickel steel. To this it has been objected that if there is sediment around the pintle it may settle into the space left by the movement and prevent the pintle from getting back, besides which the plates are liable to rust together. The latter objection could be met by using a base plate of non-rusting metal. A modification of this design is to use the ordinary type and to elongate 3 or 4 inches the recess or cup in the shoe which fits over the pintle. (See Pl. 50.) With this provision sediment can give no trouble, and the gate can move out and back but cannot move sideways. While these so-called "sliding pintles" have been rarely used, they appear to be coming into favor, and in one or two cases they are said to have proved of value.

The second type of pintle described above has been used satisfactorily for locks up to 55 feet in width, and could doubtless be used for wider ones, and possesses the merits of non-rusting and ability to stand hard wear. By chilling the surfaces of contact of the cast-iron pintle and the cast-iron shoe they will wear longer than pintles of steel or shoes provided with special bushings. Pintles and shoes of this type have been removed after eight years' service under an average traffic of between 2000 and 3000 lockages per annum, and showed practically no wear of the chilled surfaces and no deterioration from rusting. The "chilling" is secured by placing as part of the mold a cast-iron piece fitting the part to be chilled. When the metal is poured in it chills on meeting the cast piece, and forms against it a smooth dense surface, so hard that a file often cannot mark it.

The radii of the bearing surfaces of the pintle and shoe usually differ from $\frac{1}{8}$ to $\frac{1}{4}$ inch so as to prevent binding, the radius of the pintle being the smaller. The larger radius mentioned is used for flat-topped designs. There should also be a clearance or flare at each side between them, towards the bottom of the shoe, of $\frac{1}{4}$ to 1 inch, so that if the gate is torn loose it will free itself at once from the pintle without disturbing the latter. (See Pl. 50.) Pintles for very small gates are usually about 4 inches in diameter at the top, and for locks of 55 feet in width, about $5\frac{1}{2}$ or 6 inches in diameter. For locks 100 feet wide and of high lift they have been made 10 inches in diameter. In concrete locks the pintles may be placed directly on the masonry, using a unit pressure of 200 to 400 pounds per square inch, or special bearing-stones may be employed. The former method is considerably cheaper, and is the one now usually adopted.

Bonnet and Pin.—The bonnet, or connection at the top of the heel, is sometimes formed in wooden gates by using two iron straps bent to fit the heel, one at the top of the gate, the other about a foot lower, with a small casting for the gudgeon pin to which the anchor-bars are connected. (See cut of gate on p. 461.) A more usual method is to employ a casting which fits over the top of the heel, as shown on Pls. 47 and 50. It should always be provided with a movable cap, so as to permit the anchor-bars to be lifted up and replaced when required during repairs. In some of the older designs, where this was not done, it has been necessary to raise the entire gate when making repairs which required a removal of the bars.

The gudgeon or bonnet pins usually vary from about 2 to 6 inches in diameter (8 inches having been used at the 100-foot lock at St. Mary's Falls, Michigan), according to the size of the gate, the diameter being generally determined by the bending moment. They can be of steel with cast thimbles as noted in a preceding paragraph. All pins should have $\frac{1}{32}$ to $\frac{1}{16}$ of an inch clearance in the pin-holes so that they can be removed easily, as rusting in some cases and the solidifying of oil in others tend to cement them in place. Pins under water should have $\frac{1}{16}$ inch clearance.

Hollow Quoins.—For information regarding hollow quoins see p. 416 and after.

Center of Rotation.—The center of rotation of the gate should be placed a little upstream of the center line, so that the leaf will swing free of the quoin as soon as it begins to move. This distance may be ½ inch or more, according to circumstances. (See Pl. 50 and "Hollow Quoins," p. 416.)

Diagonals.—The diagonal straps used with wooden gates are sometimes made in one piece, and sometimes in two, joined by a turnbuckle. In the former case the straps are made a little short, from ¼ to ⅜ inch, and are put in place by being heated and shrunk on, and in the latter they are put on as ordinary straps and screwed tight. This is the preferable method, as should one become bent or broken by a boat or from other causes, it is little trouble to remove and replace it. No strap or other iron should be placed within 2 inches of the vertical surfaces of contact of the toes, as these in course of time may need to be trimmed if the gate has sagged, and the iron would then have to be removed and refitted.

The diagonals used with metal gates have been described on p. 469.

Lap on Sills.—The lap of the gate on the miter-sill may be made from 4 to 6 inches, and 6 inches is sufficient for the clearance between the bottom beam and the floor. Rollers traveling on a track on the floor of the recess, and intended to support the bottom of a miter-gate at the toe, have become practically obsolete, as mud or débris frequently hindered their working and rendered the support unreliable.

Miscellaneous Fittings. — The rollers on which the gate-spars travel (see also p. 482 and Pl. 50) and which may be two in number for each spar, one being placed each edge of the wall, should be of ample diameter, to minimize the friction. Small rollers soon become "gummed up" and refuse to turn. Light and serviceable ones can be formed by taking a piece of 3½ or 4-inch gas-pipe and closing the ends with cast plugs, in which are inserted short lengths of 1-inch rod to serve as journals. The journal-boxes should be arranged so that the rollers can be lifted out when desired; this is usually done by making them U-shaped, or open at the top.

If the gates may have to be opened or closed occasionally under a head, as with movable dams (see p. 486), ring-bolts of 1¼-inch iron should be provided at the toe, one at the top and one near the middle, for the attachment of lines. Means should also be provided for locking the gates together or for holding them back in their recesses, according to circumstances, during seasons of floods.

Bumping-blocks should be provided on the upstream side near the toe of each gate, one near the top, the other near the bottom. Their object is to hold the gate parallel with the walls when open, and they must therefore be of a width such that when the gate is in this position the blocks will touch the masonry. With metal gates these blocks are sometimes fastened to the masonry.

Each gate should be provided with a hand-railing, placed on the upper side so as

to be out of the way of boats. It may be made of $1\frac{1}{4}$-inch gas-pipe, with uprights of the same size, fitting loosely into cast-iron sockets bolted to the gate. On the approach of floods the railing can then be lifted out in one piece and removed.

Miter Joints.—The upstream side of the miter or toe, both in wooden and in metal gates, should be beveled off slightly for about one-third or one-half of its width, so that the actual surface of contact when mitered will fall upon the downstream portion. This is done in order to throw the compression from the end reactions into the downstream sides of the beams, thus relieving the tension existing in them from the direct loading, and at the same time avoiding increasing the compression which exists from the same cause in their upstream sides. The bevel should be slight, so that if the gates are struck from below there will have to be a considerable movement before one toe can slip by the other. (See Chapter X, "Accidents to Lock Gates.")

Alignment.—The usual practice is to place the gate in its recess so that its face will be from 6 to 12 inches back from the face of the chamber when the gate is open. This is done to prevent boats from striking it, but fenders should be provided extending about to the chamber line, or boats will strike and break off the edges of the square and of the hollow quoins. These fenders should always be placed horizontally, not vertically, as vertical ones are more easily damaged or knocked off. In other cases the gate itself is made flush with the chamber and acts as the fender. Either way is satisfactory in practice, since craft rarely cause any injury to gates when they glance against them, if the diagonal straps and other projections have been placed so as to be secured from damage.

Gates for Concrete Locks.—The heels of gates for concrete locks should be designed so that a certain amount of trimming can be secured without difficulty, as in this class of masonry it is not practicable to secure as accurate a hollow quoin surface as can be obtained in a lock of cut stone.

Life and Care of Gates.—The life of a wooden gate in the United States appears to be from 10 to 20 years, the latter being a limit which is very rarely reached. However, as the submerged part of a gate is not as subject to deterioration as the upper part, the leaf is frequently repaired by rebuilding the part above water, and retaining the submerged portion, which will usually be in a condition for renewal by the time this second top has become decayed. On the North German canals the life of a wooden gate is given as from 17 to 38 years, with an average of 25 years, the same average being given for the wooden gates at certain French harbor locks. An example was to be found in 1904 of a wooden gate at Antwerp still in good condition after 40 years of service, the timber being creosoted fir, while gates of Demerara greenheart at some of the English locks have lasted for more than 40 years, and some of those in the docks at Liverpool,

built of oak, are said to have had 70 years of service. The comparatively short life of American gates is possibly due to severer climatic conditions.

The life of iron gates used in canals in Germany has been from 28 to 48 years, with an average of 40 years, and in Holland similar gates have lasted for 60 years. Galvanized iron gates used in sea locks in France are estimated to last from 35 to 40 years.* It is to be noted that these gates were of iron, not of steel. Steel deteriorates much more quickly than iron when exposed to water. Colonel Hodges, in his work on Lock Gates before alluded to, after quoting examples of iron gates constructed between 1850 and 1870, and still in use in 1892, gives the average duration of a metal leaf as 40 years, when properly cared for and repaired.

The greater durability of wooden gates in Europe, as compared with those in America, is a factor largely in favor of their use, and metal gates, where exposed to salt water, do not appear to last much longer than those of wood. For such positions greenheart timber is frequently used, as it withstands the ravages of the teredo better than any other kind, although it is said to be subject to their attacks in tropical waters.

As the lock gates are so vital a point in a system of navigation, it is unwise to allow them to deteriorate until they become weak. On the Big Barren River in Kentucky, when that stream was under the control of a private company, a set of lock gates were allowed to go without repairs until a portion of one of the lower leaves gave way. A raft was passing out of the lock at the time going upstream, and was caught in the resulting current, and before it could be checked struck the lower gates. The timbers were so decayed that the shock broke several more of them, increasing the current till in a few moments all the four leaves were carried out, and the river was pouring through the chamber. As it was the time of low water, the damage was not as serious as might have been anticipated, but it was many weeks before navigation could be resumed.

With fixed dams of high lift it is usually advisable to open the lower gates just before a flood drowns the lock, and to fasten them securely in that position to the walls. This may cause some deposit of sediment in the chamber, but if the gates are left closed, the reaction from the dam may force them open and injure them seriously by hammering against each other and against the walls. This has happened in several cases. In one of them it had been the practice for several years to close the gates and tie them together with a $1\frac{1}{4}$-inch iron rod at the top. During one rise some unusual combination of forces occurred which tore the rod in two, thus setting loose the gates and injuring them greatly. While this procedure will not always ward off damage, since examples are known where

* Papers read before the International Congress of Navigation, Düsseldorf, 1902.

lower gates, fastened open, have been torn down by extremely violent rises, it appears to be a safer practice than that of leaving the gates closed.

Care must always be taken to see that wooden gates are not too buoyant when the water rises over them or they will float up at the toe and be liable to injure the top masonry of the quoin, and possibly displace the pintle. Cast-iron weights, preferably of about 30 pounds each, are usually employed to counteract this tendency, being placed on the arms of the gate at the approach of the high-water season, and removed when it has passed.

Operation of Gates.—For opening or closing gates the most common method in use, and probably the best one, consists of a spar provided with a cast-iron rack (see Pl. 50), and worked by a gear-wheel on a capstan, or a spar without any rack and worked by a winch or a capstan and an endless line, one part of the line pulling on and opening the gate, the other pulling on the outer end

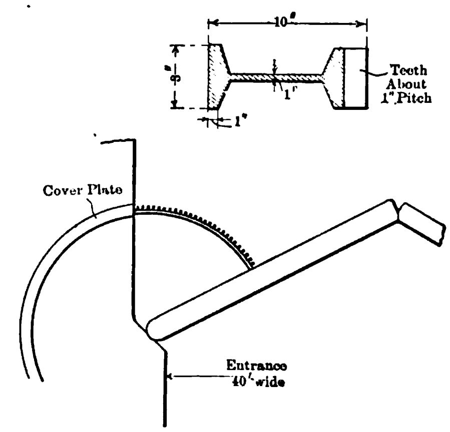


FIG. 184.—Section and Plan of a Cast-iron Arc Spar as Used on the Locks of the Lower Seine for a 40-foot Opening.

of the spar and closing it. (See illustration on p. 401). The spars move on rollers (p. 479), and as a rule are attached at a distance from the toe equal to one-third of the length of the gate, so as to utilize all the leverage available, although in many cases the attachment is made at half length, and in others, where power is available, at one-third of the length from the heel for the sake of using a shorter spar. Where the spar is long, and overhangs the wall, as it frequently does, on the river side, a light movable bracket carrying a roller is used to support the free end, attached to the wall in a way to permit its removal for an approaching flood. In some cases, as at the lock on the Mississippi at Moline, Illinois, and at Ymuiden on the Amsterdam Ship Canal, no rollers are used, but the free end of the spar rests on a carriage traveling on a winding track. Probably the simplest type of spar in use is the one shown in Fig. 184, consisting of a cast arc with teeth, attached at one-third length from the heel, and worked by gearing on the wall. This design is compact and efficient, requiring a minimum of room and

small recess in the coping. It is in use in many places in Europe, and for more than twenty years has given satisfactory service on the large locks of the lower Seine, where the traffic is heavy and the gates of considerable size.

For large gates (as at the locks of the Louisville and Portland Canal around the Falls of the Ohio River) chains have been sometimes used, attached to the bottom of the leaf, the closing chain crossing the miter-sill and chamber and passing over sheaves and up the opposite wall. The chains are operated by power. In several large locks of recent construction in Europe, as on the North Sea and Baltic Canal, the method of chains has been discarded, and a steel strut substituted, designed on the general principles of the one shown on Pl. 50 and placed above water, where it can be easily reached for repairs. It is worked in some cases by an hydraulic cylinder and in others by geared turbines or by electricity, and has been found preferable in many ways to chains, as the latter are difficult to repair, and are occasionally broken by heavily laden boats grinding over them where they cross the miter-sills.

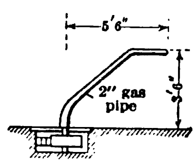


FIG. 185.

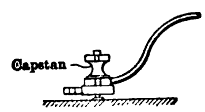


FIG. 186.

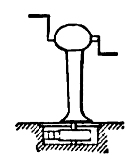


FIG. 187.

Types of Gates Spar Machinery.

The old method of the balance-beam has also been applied, in a modified form, the top beam of the gate being extended back to form an arm to the outer end of which a rope is attached wound by a capstan, or a link chain worked by sprocket wheels and levers. Recent examples of this type are to be found on the Warrior River in Alabama, and are reported to have proved quite satisfactory, but the method has not come into general use.

A satisfactory type of spar for locks up to 55 feet in width is shown on Pl. 50, and consists of two laced channels, with the cast rack bolted on in sections $2\frac{1}{2}$ to 4 feet long. For locks up to 36 feet in width 3-inch channels have been used, although some engineers prefer 4-inch channels, as they are stiffer and do not weigh much more. For locks of 55 feet width 4-inch channels should be used. (For remarks on rollers see p. 479). Two methods of operating the spar are in general use. The one most common in America is shown by Figs. 185 and 186, and consists of a lever working directly on the shaft of the pinion-wheel. Sometimes a capstan barrel is provided on the shaft as shown, worked by the same handles or levers as the pinion, and is at times very useful in moving drift, etc. The method most common abroad is shown by Fig. 187, and consists of a cast-iron standard enclosing a long pinion-wheel shaft with bevel gearing and winch

handles at the top. The American method is somewhat more direct, but is perhaps less neat in appearance.

The spar and wheel have hitherto usually been placed upon the top of the wall, but where practicable it is a good plan to place the spars below the coping, so that they will not be in the way of the lock-tenders during maneuvers. This method is in use on the Moldau in Bohemia, on the New York State Barge Canal, on the Fox River in Wisconsin, and many other places, and has proved very satisfactory. The spars work in recesses covered by cast-iron or wrought-steel plates, or by slabs of reinforced concrete.*

For the lock gates of the Panama Canal a spar is used attached to the gate at one end as usual and with the other end connected by a pin to the rim of a large gear wheel in the machinery recess. The spar is attached to the gate at a distance from the heel equal to a little more than one-quarter of the length of the gate. When the gate is closed the pin is close to the face of the recess masonry, the spar being then on a line with the center of the gear wheel. To open the gate this wheel is turned through about one-half of a revolution, carrying the end of the spar with it and pulling open the gate. The movement of the gate commences slowly, reaching a maximum when the wheel completes a quarter-revolution, and then decreasing slowly and stopping when the wheel has reached the end of its half-revolution. When at rest in this position the pin and spar are slightly past the center point. The closing of the gate is performed by reversing the operation. The main wheel is turned by a train of gears driven by an electric motor. The design is reported to work very successfully.

Latches should always be provided for holding the gates open during lockage operations.

Power for Operating Lock Gates.—No simple and practical formula has yet been given to determine the power needed to operate a lock gate. With small gates the displacement of the water takes a large proportion of the power required, but with gates of moderate or large size the friction, and sometimes the effect of wind, are the chief factors to be overcome. Below are given the sizes of spur-wheels, etc., used in certain examples, all the gates being operated by spars and pinion-wheels, turned by levers. The opening or closing under ordinary conditions could be easily accomplished by one man. The spars were attached near the ends of the gates, and the levers were 4 to 5 feet long.

Gate	12	inches thick,	15	feet	9	inches long,	22	feet	0	inches high,	pitch diameter of spur-wheel	$11\frac{1}{2}$ inches.
"	15	"	21	"	9	"	29	"	6	"	" "	$8\frac{19}{32}$ "
"	18	"	30	"	9	"	34	"	0	"	" "	$8\frac{19}{32}$ "

* On the upper Kentucky River, where this method is in use, it has been found difficult with a rapidly-rising flood to remove the spars before the walls become submerged, owing to the necessity for removing the recess covers first.

All the gates were of timber, the two last being paneled. Where the gates are of metal they may require less power, being usually lighter than corresponding sizes in timber, except where the latter are largely immersed.

Experiments conducted in Canada to determine the horse-power, etc., needed to move lock gates gave the following results: A solid wooden gate 18 feet high, 23 feet long, moving in 10 feet of water, and with the spar attached about three-fifths of the length from the heel, required a direct pull or push on the spar of 1000 to 1200 pounds at starting, and 500 to 600 pounds to keep moving. The time of operation was one minute. Motors of 2 horse-power each were adopted. Another gate, also of the solid wooden type, 31½ feet high and 27 feet long, moving in 15 feet of water and with the spar attached 23 feet from the heel, required an initial pull of 3000 pounds, and 1200 to 2000 pounds afterwards. The time of opening was one minute, and the net horse-power used (not allowing for friction of machinery) was about 0.7. For gates similar to the preceding, but about 42 feet in height and with the spar attached one-half of the length from the heel, motors of 3 horse-power each were adopted, with a provision for 25 to 50 per cent overload for initial motion, and proved very satisfactory. On other gates somewhat lower in height, but of older design, motors of 5 horse-power each, were used, so as to provide ample power.

For the steel gates of the 20-foot lift lock at Munster on the Dortmund-Ems Canal, with a chamber width of 28¼ feet, and 10 feet on the sills and spar attached about one-third from the heel, motors of 5 horse-power are used. The turbine is about 15 horse-power, and (as on the River Weaver in England) is operated by the same water as used for lockages, storage batteries being used. At the Poses lock of the Lower Seine with gates about 30 feet high and 25 feet long, moving in about 11 feet of water and with the spar attached one-third from the heel and of the type shown in Fig. 184, p. 482, motors of 3 horse-power are in use. The same motors operate, by a connection through a clutch and gearing, the large butterfly valves used for filling and emptying the chamber. The turbine generating the electricity varies from 12 to 30 horse-power, according to the head. At the Horin lock on the Moldau, with heavy steel gates and an entrance width of about 40 feet, 2 horse-power motors are used for each upper gate and 4 horse-power for each lower one, the spar being of the same type and attached as at Poses. The turbine is of 31 horse-power.

At the Canadian lock at Sault Ste. Marie the electric motor used to operate a leaf about 37 feet by 40 feet is of about 25 horse-power, and at the lock at the same locality on the American side a hydraulic motor of approximately the same horse-power is used for a metal leaf about 43 feet by 50 feet. At the Cascade locks of the Columbia River, Oregon, a metal leaf somewhat similar in size was found to require about 10 horse-power for its operation under ordinary

conditions, and at the large lock at Bougival on the Seine, near Paris, the hydraulic motor (consisting of a cylinder and piston rod attached directly to the gate) for operating a wooden leaf about 39 feet by high by 26 feet in length can exercise a direct force of 14,000 pounds.* The spars are attached about 6½ feet from the heel. For steel lock gates on the New York State Barge Canal, with a length of about 24 feet and an extreme height of about 50 feet, the maximum pull on any gate spar, allowing for wind, etc., was taken as 11,000 pounds. The spars were attached at one-third of the length from the heel. The amount of power required in a given case will of course vary with the method of operation and the point of its application.

The gates of the Elbe-Trave Canal are operated by counter-weights and compressed air (see p. 449), and those of the lock at Bremen by spars with a cable attached to each end. The cables are pulled by pistons moving in vertical wells in the lock walls, the movement being obtained by admitting water on to the upper side of the pistons.

The spars and operating gear for river locks should, where exposed, be all designed so that they can be detached without difficulty and carried out of danger from high water. Where this has not been done, damage from drift or ice has frequently resulted.

Means for operating the gates by hand power should always be provided for use when the machinery gets out of order. (See also "Power for Operating Valves," p. 105.) In the United States power is rarely used except where the gates are large or the traffic is very heavy, and gates of locks with chamber widths ranging up to 55 feet and lifts up to 18 feet are operated by hand without unusual difficulty, one man usually being sufficient for the maneuvers, except where the locks approach the limits just mentioned.

Opening and Closing Lock Gates under a Head.—It is sometimes necessary where movable dams are used to open or close lock gates under a small head, the practice being to open the lock gates when the dam is lowered for the winter, so that deposits of mud can be washed out in the spring. With rivers carrying a small amount of silt this practice has been gradually abandoned, as with the movable dams of Upper Seine, where experience showed that no objectionable silting occurred if the gates were kept closed. On American rivers, however, the gates have usually to be opened, as the deposits are often large. With fixed dams the lock walls are rarely submerged for any lengthy period, and therefore a continuous depositing of silt between them is unusual, and little trouble occurs. With the lower height of lock wall used with movable dams, however, there is generally found a considerable deposit of mud and débris in the chamber after a long submersion, and if the gates have been left open this can be easily washed out by raising the dam. Cases have occurred where such deposits have filled parts

* Annual Report, Chief of Engineers, U. S. A.,

of the chamber to within a few feet of the coping, and where, had the gates been closed, it would have been impossible to open them without great labor and delay. Another reason for opening the gates is to allow the river to use the additional discharge area of the chamber. This feature, however, is not often an important one.

On the Ohio River it is customary to open the chamber for every considerable flood. The gates are of the rolling type, built of steel and closing a chamber 110 feet wide, and are opened or closed across a 2 to 4 miles per hour current without special difficulty. On the Kanawha River wooden gates about 33 feet long (closing a 55-foot chamber) are pulled open under a 6-inch head and in a depth of water of 16 feet. Triple blocks using $1\frac{1}{4}$-inch line are attached to a snubbing post and to the toe of the gate at the top, and 4 to 5 men work the spar capstan and pull the gate open. To close it one man passes the line round a snubbing post and slacks it off as needed. These gates have at times been opened under a 12-inch head. On the North Sea-Baltic Ship Canal the gates of the guard lock at the Baltic end are sometimes closed under a head of $1\frac{2}{3}$ feet. The chamber is 82 feet wide and the gates are provided with large sluices which are left open during the operation so as to reduce the water pressure. At Lock No. 2, Big Sandy River, gates spanning a chamber 55 feet wide have been closed under a head of 6 feet by using lines attached to the top and bottom of the gate. The gates are usually left open to facilitate the placing of the large needles of the dam, and by the time this operation has been finished the head of water has usually increased to considerable proportions. However, closing gates under such a head is attended with some risk, and is, beyond doubt, detrimental to the gate in the long run.

Where gates are opened and closed under a head of 6 inches or thereabouts, no harm has been found to result from the tendency to twisting or wrenching, as the framing has sufficient elasticity to resist it. Where the head is increased to a foot or two, however, lines at top and bottom, or a single line near the center of pressure, should be used. Great care must always be taken to see that the tackle and posts are amply strong, as if control is lost the gate may crash against the miter-sill with sufficient force to cause a breakage, and in extreme cases the gate will "jump" the sill and tear loose from its anchorage.

Effect of Ice on Lock Gates.—As far as experience goes there appears to be little danger of injury to lock gates, either of wood or of steel, from the pressure or the gorging of ice. Thus on the Allegheny and Monongahela rivers, which bring out a considerable amount of ice during the winter and spring, cases have occurred where the ice has gorged in cakes 2 feet in thickness to a height of 20 feet above the lock walls, completely burying the gates. It was found subsequently that no injury of any kind had been sustained by them. These gates were built of timber. Experience with steel gates under such conditions is limited,

as yet, but there appears to be no reason why they should not be capable of withstanding the effects of ice at least as well as gates of wood.

Where there is danger from ice the gate spars and all operating machinery projecting above the wall and in a position liable to injury should be made removable. Similarly the railing on the gates should be removable and the gates should have as few projecting parts as possible which might be bent by pressure or impact.

Cost of Gates. *Wooden Gates.*—The cost of gates varies considerably, as will be seen by the following figures, which give the cost of several wooden gates built by the United States Government by hired labor.

Two pairs of gates of 12-inch timbers, for a lock 27 feet wide and of 9 feet lift built in 1896:

Cost of timber (13,000 feet B. M.)	$325.00
" cast iron (14,000 lbs.)	423.50
" wrought iron (9400 lbs)	553.50
	$1302.00
Framing, placing, and miscellaneous	1204.00
(about $92.50 per M.)	
Total	$2506.00

About $193 per M., or $2.20 per square foot.

(This includes cost of operating spars, butterfly valves, etc.)

Two pairs of gates, of 15-inch timbers, for a lock 36 feet wide and of 14 feet lift, built in 1899:

Cost of timber (29,000 feet B. M.)	$690.00
Cast and wrought iron	824.00
	$1514.00
Framing, placing, and miscellaneous	1406.00
(about $48.50 per M.)	
Total	$2920.00

About $101 per M., or $1.45 per square foot.

(These had no valves in the gates. The cost of the operating spars included.)

Two pairs of gates, 18 inches wide, of two 9-inch timbers, keyed, for a lock 52 feet wide and of 15½ feet lift, built in 1897:

Cost of timber (57,600 feet B. M.)	$944.00
Cast and wrought iron	634.00
	$1578.00
Framing, placing, and miscellaneous	2315.00
(about $40.25 per M.)	
Total	$3893.00

About $67.60 per M., or $1.32 per square foot.

(These had no valves in the gates. The cost of the operating spars is included.)

It should be noted that the price of timber and other materials has increased greatly since the foregoing gates were built.

The cost of wooden gates on the North German canals, for heights of 30 feet and widths of chamber of 42 feet, is stated to be about $3.60 per square foot, and for double-sheathed iron gates of the same size about $4.50 per square foot, or one-fifth more.

The cost of wooden gates at certain French harbor locks is given as $5.50 to $7.00 per square foot for widths of chamber of 42 feet to 52 feet, and the cost of iron gates for widths of chamber of 69 feet as $8.20 per square foot.

On the Manchester Ship Canal, with a depth of water of 40 feet and a width of chamber of 65 feet, the cost of greenheart gates was about $41,000 per pair, the estimate for iron gates of the same size having been about $28,000 per pair, or one-third less. Greenheart timber was used in preference to iron on account of its superior durability.*

Steel Gates. (See also paragraphs just above.)—The cost of several sets of steel gates, erected between 1898 and 1902 on certain tributaries of the Ohio River, ranged from $2.70 to $3.20 per square foot in place, the heights being from 27 feet to 33 feet, and the corresponding lengths from 30 feet to 20 feet for each leaf. The weights per square foot varied from 45 to 70 pounds. The weights of steel gates on a 52-foot lock on the Kentucky River with an 18-foot lift are: Anchorages 10,500 pounds; 2 upper gates, total 47,000 pounds; 2 lower gates, total, 123,000 pounds; operating machinery and appliances 5200 pounds. The cost ranged in 1912 from 6 to $7\frac{1}{2}$ cents per pound, erected.

The contract price of the five pairs of double-sheathed arched gates for the lock of the St. Mary's Falls Canal, Mich. (1893), was $182,610. The steelwork of the main framing was 5.9 cents per pound; the cast iron, 6.0 cents; the cast steel, $9\frac{1}{4}$ cents; the steel forgings, $27\frac{1}{4}$ cents, and the bronze, 35 cents. Of the first-named item there were 2,405,000 pounds, and the others a total of 169,300 pounds. The width of chamber is 100 feet, with 21 feet of water on the sills, and a lift varying from 18 to 20.8 feet, the largest gate being about 43 feet high, and the smallest about $25\frac{1}{2}$ feet. The cost per square foot was approximately $13.

On the German canals just alluded to it was the practice to use wooden gates for locks up to 42 feet in width, and iron gates for greater widths. In Holland the use of wood has been limited as a rule to widths of chamber of 60 feet. Since 1900, however, the use of metal gates abroad has greatly increased, and the type appears to be supplanting the wooden gate.

The contract price of steel gates of the pure arch type, designed about 1907 but not used, for the New York State Barge Canal was as follows: Gates 20 feet high and $25\frac{1}{4}$ feet long weighed 37,000 pounds each, and were to cost $2200. This

* Papers read before the International Congress of Navigation, Düsseldorf, 1902.

was equivalent to a weight and cost per square foot of 73.3 pounds and $4.36 respectively, and a price of about 6 cents per pound. They were designed for a depth on the sill of 12 feet, a lift 15 feet, and a width of chamber 45 feet. Similar gates (for the lower end of the locks) 31 feet high weighed 58,000 pounds each and were to cost $3300 at about 5.7 cents per pound, equivalent to a weight and cost per square foot of 74 pounds and $4.21 respectively. Gates 45 feet high and 25¼ feet long, for lift of about 27 feet, weighed 80,000 pounds each and were to cost $5600 at 7.0 cents per pound. This was equivalent to a weight and cost per square foot of 71 pounds and $4.97 respectively. The prices were for gates set up in place, and the weights are those of the gates proper, without anchors, etc. The type was later changed to the ordinary mitering girder gate, and a reduction of one foot made in the length. The contract prices for the gates set up in place were then as follows: Gates 20 feet high and 24¼ feet long weighed 44,500 pounds each and cost $1980 at about 4½ cents per pound. This was equivalent to a weight and cost per square foot of 91½ pounds and $4.06 respectively. Corresponding gates 31 feet high weighed 66,000 pounds each and cost $2942 at about 4½ cents per pound, equivalent to a weight and cost per square foot 87.6 pounds and $3.90 respectively. The gates of 45 feet height weighed 97,000 pounds each and cost $6260 at about 6½ cents per pound, equivalent to a weight and cost per square foot of 90.6 pounds and $5.85 respectively. All weights are for the gates without anchors, etc. Each end of each gate was curved on the upstream side, being generally similar to the type recommended on p. 475.

CULVERTS AND VALVES

Methods of Filling and Emptying.—The filling and emptying of the chamber is regulated by valves or wickets placed in the lock gates, or in culverts in the masonry. Sometimes the combination of culverts for filling and valves in the lower gates for emptying, and vice versa, is employed. On rivers which carry much sediment the discharge from emptying should whenever practicable be made into the tail bay, as this will assist in keeping the entrance free from deposits.

Culverts.—Culverts may be divided into four general types: miter-wall culverts, longitudinal culverts, transverse culverts, and floor culverts. The short culverts often used around the lower gates may be considered as a modification of the second type.

The first type is generally used for filling. The water enters through vertical openings in the upper gate recesses or through horizontal ones in the upper miter wall, and falls into a culvert which crosses the lock just above the gates and in the masonry of the wall. (Figs. 188 and 189.) From this several smaller outlet

culverts pass at right angles into the chamber. The direction of the discharging currents is practically similar to that of valves in the gates, but the outlets are usually lower and much larger in proportion to the volume, and it has accordingly been found in practice that boats are troubled much less by the resulting

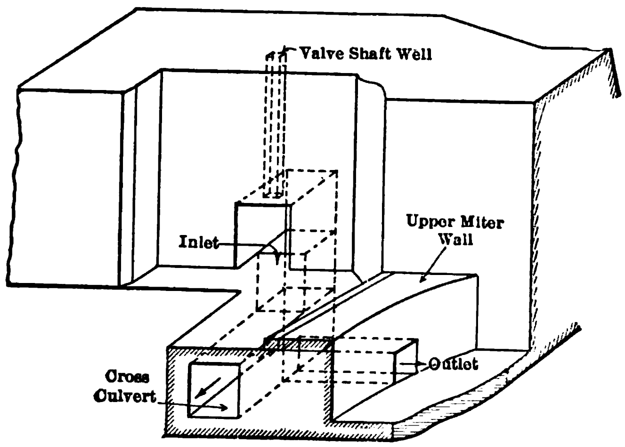


FIG. 188.

currents than with valves in the gates. This type has been largely used in the United States for locks of moderate size and with high upper miter walls. It has rarely been used for emptying, as cases where the lower miter wall is high enough to permit its application are exceptional. The emptying is usually done either by valves in the gates or by short culverts opening in the lower gate recesses, passing

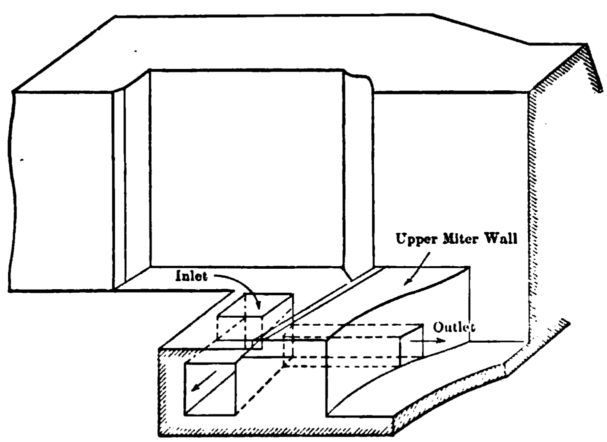


FIG. 189.

around the heel of the gate, and discharging into the tail bay immediately below. (Fig. 190.)

The second type—the longitudinal culvert—has come into much favor both in America and Europe, especially for large locks. It opens as usual in the upper gate recess and passes down and inside the chamber walls, emptying into the

tail bay. Along the walls and close to the floor (Fig. 191) small openings or "ports" (spaced usually from 20 to 30 feet apart and of total area from 30 to 50 per cent greater than that of the culverts so as to lessen the velocity of discharge) pass from the culvert to the chamber, spaced about uniformly and opposite each other.* Through these openings the lock is filled and emptied and the distribution of the current is such that all dangerous effects upon the boat are avoided. While

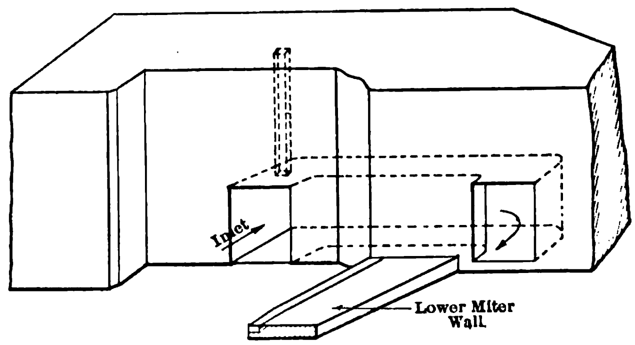


FIG. 190.

this type occasions some additional expense, as it requires wider chamber walls, it appears to give the most satisfactory results in operation, and the general principle should be employed wherever heavy boats are to be handled. A modified arrangement is sometimes used in which the culverts run only part way down the lock and serve for filling only, the emptying being done by valves in the lower gates, or through culverts around them, as in Fig. 190.

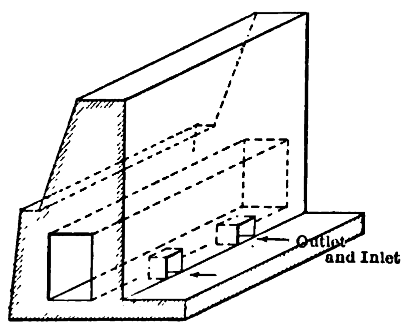


FIG. 191.

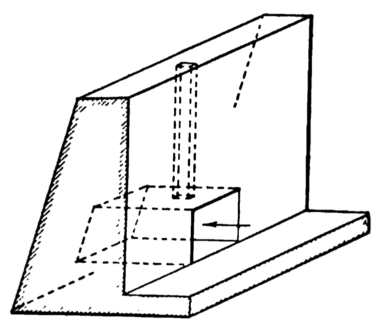


FIG. 192.

The third type—the transverse culvert—has been applied to the locks of the Ohio River and to occasional ones elsewhere. On the Ohio locks a series of openings (16 for filling and 16 for emptying) is placed in the river wall, as shown in Fig. 192, above and below the dam. These are closed by cast-iron butter-

* At the Horin lock of 30 feet lift on the Moldau, the ports were originally built opposite to each other, but were later on changed and "staggered." The alteration was said to reduce the movement of the filling currents considerably. At the Little Falls lock on the Mohawk River, N. Y. ($40\frac{1}{2}$ feet lift), the ports are also staggered. On the other hand, there has been no trouble with the high-lift locks of the Soulanges Canal in Canada, in which the ports are opposite.

fly valves, $4\frac{1}{2}$ feet in diameter, moving on vertical shafts. The water enters the chamber directly from the river through one set of valves placed upstream of the dam and empties directly into the river through a second set of valves placed downstream of the dam. The method has worked very satisfactorily, but it was found in the earlier examples that where the filling valves were some distance downstream of the upper end of the wall, the slope of the river in moderate stages resulted in a level in the chamber slightly lower than that above the gates, and the head had to be equalized by using valves in the gates before they could be operated easily. There is some tendency also, as with valves in the gates, to an uneven flow of water in filling the chambers.

In the fourth type—the floor culvert—the water usually enters just above the upper gates and passes down and along in culverts under the floor and out under or behind the lower gates. The roof of the culvert is provided with openings through which the water enters or leaves the chamber. In certain cases, as where the lock is set into solid rock, requiring only a facing to form the sides of the chamber, the type may be found economical. So far it has rarely been used, probably the most well-known example in this country being at the locks at St. Mary's Falls, Michigan, through which passes most of the east- and west-bound commerce of the Great Lakes. The culverts in the larger lock are six in number, running from above the upper to below the lower gates, and with 214 openings into the chamber. (See illustration on p. 395.) Provision must be made in designing such culverts to guard against the tendency of the water to force up the roof when filling under a full head. This method of distribution has proved very satisfactory and produces little objectionable disturbance in the chamber, although in long locks the water flows up through the upstream outlets several seconds in advance of those further down the chamber. Where desired the emptying may be done by connecting with culverts around and behind the lower gates (Fig. 190), as described for the first type, instead of continuing the main culverts under the lower miter wall.

Valves in Gates.*—Valves in the gates are probably the cheapest and simplest type and have been used successfully in the United States for locks of lifts up to 18 feet, the size of the valves being decreased in inverse proportion to the lift, so that one man can always handle them. One objection to them when used for filling is that in high lift locks the valve is at first above water, and the filling has to be carried on slowly until the lower level has drowned the valves. If this is not done, the discharge is liable to pour on the bow of the boat. This arrangement, however, is now rarely seen except in old locks, as in those of later construction other methods of filling have been used to meet such conditions. Another objection, which applies to locks of any lift, is that the water pours

* Cuts of valves will be found on pp. 461 and 462, and on Pls. 51, 52, 52a, and 53.

in from one end only and in concentrated volume, creating a lengthwise reversal of current which tends to drive the boat against the upper gates, and requires a very careful handling of the lines. It is also necessary to begin filling slowly, as the less water there is in the chamber, the more violent is the effect of a full opening of the valves. For these reasons, while this type has been used successfully for many years on rivers and canals handling large traffic, as on the Kanawha River locks in West Virginia with lifts of 8 feet and chambers 55 feet by about 340 feet, and on the ship canals of the St. Lawrence River with lifts up to 14 feet and chambers 45 feet by about 260 feet, the trend of modern design is towards filling through culverts.

Few practical objections exist to using valves in the gates for emptying. They can in most cases be opened fully and all together, and produce a slow movement of the water in the chamber, without the dangerous effects upon the boats such as appear during filling. Many locks are in satisfactory use which are filled by culverts passing under the upper miter wall and emptied by valves in the lower gates.

Balanced or Butterfly Valves or Wickets.—A common and satisfactory type of valve for low and medium lifts is the balanced or butterfly valve. When used in lock gates they usually turn on a horizontal shaft, and are worked by a lever and rack wheel from the top arm. (See Pl. 51 and illustration on p. 461.) They are easily repaired and not expensive. Another type turns in a vertical plane and is worked by a rod which runs through the valve and to the top of the gate, where a simple cranked lever is used for operating, or a wheel and sector gear (see illustration on p. 462). The valve with cranked lever is simpler in mechanism than the former, but as the leverage obtainable is small, the valve also has to be made small. The following sizes of gate valves with horizontal shafts have been taken from examples of successful practice.

For heads up to 8 feet, each valve was 4 feet 3 inches long by 2 feet 5 inches wide.

For a head of 16 feet, each valve was 2 feet 6 inches long by 2 feet 3 inches wide.

These dimensions are about the extreme limits of size which can be operated for the respective heads by one man, and sizes for intermediate heads can be determined accordingly. The type is not desirable, however, for heads exceeding 8 or 10 feet, as the valves become difficult to handle unless made very small. (See also preceding clause, " Valves in Gates.")

The valves may be of cast iron or of $\frac{3}{8}$-inch steel plate, stiffened. The latter material, however, has in some instances proved unsatisfactory, owing to quick corrosion and to the fact that the rivets have broken under the shocks of closing. On some of the Kentucky River locks this type of steel valve has lasted less than ten years.

Circular butterfly valves are used on the Ohio River locks, as described on pp. 492-3.

Where butterfly valves of rectangular outline are placed in culverts they are usually made a little higher than wide, and turn on vertical shafts which connect with the operating gear, set in recesses in the coping. (See Pl. 52.) They may be of cast iron, or of ½-inch steel plate stiffened with angles or beams. Cast-iron valves of this class have proved fairly satisfactory in use, but several cases have occurred where the valves have broken, sometimes through a piece of drift jamming them, sometimes for no apparent cause except that of the flaws inseparable from this material. They have also to be made much heavier than a steel wicket. On the other hand they resist the effects of rust and foul water far better than does the steel.

The shaft for a wicket of about 25 square feet of area should be 3½ or 4 inches in diameter. If a cast wicket is used, the shaft should be forged square for the connection, not hexagonal, as with the latter shape the edges sometimes wear off, and the grip is lost. It should be slightly smaller at the bottom of the wicket than at the top, or wedge-shaped, so as to permit of its being withdrawn for repairs. The bearing of the shaft should be continuous throughout the length of the wicket as far as practicable, experience having shown the need for ample bearing surface in order to transmit the heavy torsional strain. The journals and bearings of large valves should be hardened or bushed and be of ample length, as they are liable to wear rapidly, allowing the valve to leak or causing difficulty in operation. In one case where the valve was set on a vertical shaft, the bearings wore down after ten years' use until there was an opening of an inch along the top. In another case valves which could originally be operated by one man required two or three men later on because of the wearing of the bearings. These difficulties form one of the most serious objections to the use of butterfly valves, except where they are of small size, as in lock gates. (See also "Details of Construction," p. 499.) The wicket itself is set in a rectangular cast-iron frame, which is shaped to suit the masonry and provided with ribs projecting about an inch, against which the edges of the wicket close. These ribs should be provided with wooden strips bolted on, as this permits of trimming to a closer fit than can be obtained with metal only. The arms of the wicket may be made equal on each side of the shaft; sometimes the arm which opens inward is made a little longer than the other, on the principle that as soon as the opening begins the water will act on this additional length and assist in the maneuver. While this principle is doubtless correct, it appears to be of little practical use, as the violent action of the currents in the culverts is so uncertain in different cases that unequal-armed wickets have been known to open automatically in one lock and close automatically in another, although the general design was not greatly different in the two cases. Under these con-

ditions it is believed to be best to use equal arms and to provide gearing powerful enough to permit one man to control the opening or closing at any point without unusual effort.

In one example, on the Kentucky River, where the wicket was $4\frac{3}{4}$ feet wide and 6 feet high, with a head of $15\frac{1}{2}$ feet (measured from the bottom of the wicket), the pitch diameter of the large gear-wheels was 39 inches and of the pinions $7\frac{13}{16}$ inches. The ratio of the power at the end of the operating lever to the turning force delivered at the circumference of the wicket shaft was 1 to 600, friction being neglected. Under the full head one man could operate it, but with difficulty, but with the head reduced to about 13 feet the operation was easy. On other locks with similar conditions, but less powerful machinery, two or more men were needed for the maneuver, and where additional help was not available, serious injury sometimes resulted to the operator because of the wicket getting beyond his control. It is much better, therefore, to have an excess of power than to have a deficiency. The fit of the gear-wheels on their arbors and on the shafts should not be a "machine-shop," but should be a little looser, or it will be very difficult to get them apart for repairs.

An arrangement of butterfly valves sometimes met with places the valves at culvert inlets in the floor of the upper gate recess (Fig. 189) at such a level that the lock gates clear them at all times. They are worked by sprocket chains or by cranks and levers from gearing on the walls. The type is not in general desirable, as the pieces are not easy of access, and there are more moving and breakable parts under water than with the butterfly valve which turns on vertical shaft.

On some of the locks of the Illinois and Mississippi Canal (1906) filled by culverts passing behind the upper gates, butterfly valves were used and placed flush with the face of the gate recess masonry. This made them very accessible for repairs. In the lock built at Bremen about 1908, and with similar culverts, the valves were similarly placed but were of the lifting type and counterweighted. They were opened by admitting water to the surface of pistons moving in vertical wells in the lock wall, the water causing the piston to sink and pull up the valve by the connecting cable.

Sector Valves. A type occasionally met with (as at Horin Lock on the Moldau) is the sector or Taintor valve, a description of which will be found in the paragraphs on "Sluiceways and Drift Chutes" (p. 524). This valve requires an unusually large opening in the masonry, and although simple in idea, appears to possess no great advantage over the more customary types.

Gridiron Valves.—Another style of gate valve is the gridiron or sliding valve, in which the valve is pierced with rectangular openings and slides up and down upon a frame pierced with similar openings through which the water passes. It is usually worked

by a rack and worm, but it is an undesirable type, as it is slow in operation and small in area of discharge, although permitting but little leakage.

Cylindrical Valves.—This valve, a modified form of the valve known in France as the Fontaines valve, from the name of its inventor, and sometimes called the drum valve, consists of a cylindrical ring 4 to 6 feet in diameter and about 18 inches high, sliding up and down in a second ring above, the latter having a closed and water-tight cover. (See Pl. 53.) The culvert, which has a circular orifice, opens under the ring, and the latter rests when closed upon the edges of this opening, cutting off the water. To permit the passage of the water the ring is pulled up by machinery on the wall and slides up in the second ring, thus uncovering the culvert. The moving joint is made watertight by a packing ring of rubber, and counterweights are used to balance the load. The height to which the valve should lift should not be less than one-quarter of the diameter of the culvert, in order to secure the full capacity of the latter. This is probably the easiest valve to operate, as the water-pressure balances itself on all sides, and it is suitable to any lift, but the metal work is costly and requires large openings in the masonry in order that the water may have a free approach all around, and it is not easy to replace or repair. Examples of the type are to be found on American rivers on the Muskingum, the Kentucky, the Big Sandy, and elsewhere, and it is largely used abroad. The center-wall culverts of the Panama Canal are equipped with similar valves.

This type can also be arranged with some extra expense so that the water-pressure will operate it, and some ingenious patents have been devised to accomplish this end. Valves of this modified design are in use at the St. Andrews lock on the Red River, Manitoba,* and also at Locks 1 and 2, Big Sandy River, in West Virginia. (See Pl. 52*a*.) They have proved very satisfactory in the former case, where the water is comparatively clear, but in the latter cases the large amount of sediment, as well as the drift, leaves, etc., has caused trouble with the pilot valves and the design has not worked entirely satisfactorily. This modified valve is operated by the head due to the difference in the pool levels by moving a small pilot valve (enclosed within the main valve) which regulates the admission and the release of the head within the casing. When the main valve is subjected to a head and the pilot valve is raised, the water enters from the upper pool and the pressure becomes the same on both sides of the roof. (Fig. 2, Pl. 52*a*.) The inner face of the curved lower part of the main valve is then not under any pressure, but the outer face is subjected to that due to the difference of the pools, the vertical component of which causes the valve to rise. The valve is counterbalanced, and its weight for a 63-inch opening is about 3300 pounds. The time of operation is 15 seconds.

An improvement suggested for the ordinary type of cylindrical valve is to put the closed cylinder on the inside and the open sliding cylinder on the outside, making

* See Engineering News, vol. 64, p. 351.

the former stationary, as shown on Pl. 53. This would simplify the construction, and would do away with the necessity of water-tight pipes for the valve-rod.

Another modification, and one which has much merit, is to omit the outer ring, making the valve proper as a long steel tube open at the top and bottom. The tube when lowered rests over the culvert opening, and its top projects above the upper pool level so that the water pressure balances on all sides. It is raised and lowered in the same way as the ordinary drum valve. This type is much simpler in construction than the type in general use, its operation is satisfactory, it has no parts difficult of access, and can be easily removed. Examples are to be seen at the 20-ft. lift locks at Münster on the Dortmund-Ems Canal and at the 32-ft. lift lock at the entrance of the St. Denis Canal in Paris, and elsewhere.

Roller or Gate Valves.—Valves made of gates moving on rollers (Fig. 193) have been largely used in recent construction, especially for high lifts. They are usually made of a flat rectangular steel plate, $\frac{3}{8}$ to $\frac{1}{2}$ inch thick, suitably braced, and moving on rollers which are either fastened to the valve or move separately. The latter type is known as the Stoney gate, and is described in Chapter VIII. If the rollers are fastened to the valves they should be made as wheels of $1\frac{1}{2}$ or 2 feet diameter, so as to provide for easy movement.

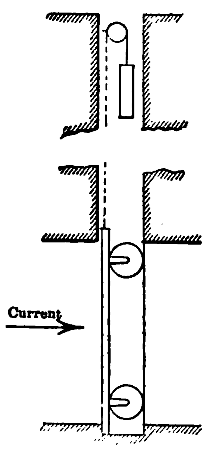


FIG. 193.

These valves move vertically in rectangular wells or openings in the masonry, being operated by machinery on the lock wall, and are usually provided with counterweights for ease of movement. On the Moldau are examples of horizontally-moving roller valves, but this adaptation is rare. The valve should be placed preferably on the upstream side of its well, since there is less chance of drift catching in the rollers if they are on the lower side, and as the journals wear there will be no tendency for the valve to grind against its frame. Valves placed on the downstream side, if closed under a head, will produce a surging of the waters in the well which may flow out on the wall.

Roller valves with rollers attached to the framing are in use among other places on the Moldau and on the Barge Canal of New York State (with lifts on the latter up to $40\frac{1}{2}$ feet), and Stoney valves are in use on the Manchester Ship Canal and on the Panama Canal, where each one controls an opening measuring 8 by 18 feet. Some of the valves on the latter canal are designed to operate under a head of 60 feet, and to withstand a head of 80 feet without undue leakage. The maximum static pressure on any one is 314 tons; the operating pressure may reach 281 tons on the lower valves. The vertical range of motion is 18 feet. The moving part of each valve weighs 22,500 pounds in the air, and the material is nickel-steel, with steel castings.

Stoney rollers to secure uniform action should always be made of cast iron or hard

steel, with solid treads of equal hardness. Wrought iron or other comparatively soft material will wear rapidly.

Choice of Type.—(See also preceding descriptions.) The various types of valves just described and methods of filling and emptying have all been in satisfactory use for many years, and the engineer will find that any one of them will prove successful, with proper designing, although all are not equally desirable, as a rule, for a given case.

For small locks with low lifts, valves in the gates are the most suitable for reasons before given, and in such cases will prove economical and satisfactory, provided the filling is done carefully. For small locks with high lifts, and locks of moderate size with either high or low lifts, culverts as in Figs. 188 and 191 can be used, with valves in the lower gates for emptying or culverts as in Fig. 190.

For large locks with either high or low lifts, where the boats will presumably be large and difficult to control if there is much movement of water in the chamber, the filling and emptying should distribute the currents as much as possible, and culverts placed in the walls as in Figs. 191 or 192, or under the floor as before described, should be used.

The most satisfactory valve for culverts is probably the vertically-lifting roller valve, either of the Stoney type or preferably of the type with the rollers attached (Fig. 193.) The latter has the merit of greater simplicity and if in need of examination or repairs can usually be lifted directly out of its well and be replaced without the need of a diver or of unwatering the recess. This is a very great advantage. Where counterweights are used—and these can be of concrete if economy is desired—the power required to lift the valve is very small, and the gearing is consequently inexpensive. One man can operate it without difficulty. This type is also free from the wearing of the journals which has been found to accompany the butterfly type shown on Pl. 52, especially on sedimentary rivers, and which is usually a difficult matter to repair, as it requires unwatering the valve chamber and taking apart more or less of the construction. It can also be lifted out of any sediment which may have settled against it during a flood, and which has at times caused trouble when opening a valve of the butterfly type. The long drum valve referred to on p. 498 as being used at Münster has also the merit of easy removal, but the arrangements for culverts and recesses are less simple than for the roller valve.

Whatever type may be chosen, its framing if partly buried in the masonry should be made as far as practicable of cast iron on account of its non-rusting qualities, and all parts should be made so that they can be easily reached or removed for inspection and repairs.

Details of Construction.—(See also preceding paragraphs.) In designing culverts or valves, the important rule should be followed which applies to all construction, and especially to river and canal work: Make all parts as simple and as easy as possible to construct and to repair. The latter point is especially important. The valves are

one of the most vital parts of the structure, and accidents to them may seriously delay or entirely stop navigation. Hence they should be of a design of which duplicates can be obtained without difficulty; they should be all alike in size and details on the same waterway as far as practicable; and they should be easy to take out and replace, as should also their frames and all fittings. Horizontally-turning valves in the gates, if well designed, can usually be replaced by a diver in a very short time, as they turn in journals or sockets purposely left open (see Pl. 51). Experience has shown that this type of valve need not be fastened in the sockets. Those in culverts are more difficult to deal with. The butterfly type, moving on vertical or horizontal shafts, usually gives more or less trouble from wear in the journals, depending on the amount of grit in the water. The journals should be bushed, and some engineers state that hard Babbitt metal gives the best results. The length of bearing should be two or three times the diameter of the shaft. Vertical shafts also wear at the bottom, sometimes seriously, owing to their weight; where practicable this should be carried by a removable collar on top of the wall, which can be examined easily and be replaced when worn. A washer should also be used where the valve rests or turns on the surface of the bottom frame, so as to carry the wear from the weight of the valve. In order to facilitate the replacing of this type of shaft, which must be done by letting it down vertically from the lock wall, it should be tapered where it fits in the valve. (See Pl. 52.)

All bolts should have square heads and nuts, as it is difficult for a wrench to get a tight grip on hexagonal nuts when the bolts are rusted fast, and this often takes place in a very short time. The use of steel and wrought iron under water, except in parts which can be easily taken out and replaced, should be avoided wherever practicable, as the metal usually rusts quickly, especially where the water is tainted with sewage or mill wastes. Cast iron appears to be the only metal (except bronze, etc.) which lasts well under water, although nickel steel appears to be satisfactory under certain circumstances, and has been used in locks gates on the Monongahela River, where the water contains a large percentage of acids.

The entrances to all filling culverts, where there is danger of trouble from drift, should be protected by screens of iron bars, hinged on the downstream side to open like a gate and latched to the wall. The bars can be about 3 or $3\frac{1}{2}$ by $\frac{3}{8}$ inches, spaced $2\frac{1}{2}$ or 3 inches center to center. These screens are needed to prevent débris becoming entangled in the wickets and are usually set against the face of the wall, but if preferred they can be set in a recess in the culvert entrance. The former method permits of easier placing and removal, and also of easier cleaning. At certain seasons leaves collect against these screens to an amount which seriously affects the passage of water, and unless promptly cleared away the pressure against them may bend the ironwork. For easy removal of such trash the entrances to the filling culverts should be placed with their tops say a foot or two below the normal water level. The screens also should have a bearing all around, as in some cases when partly clogged they have become

twisted by the unequal pressure. Some engineers design them so that they can withstand the pressure of the full head, and thus not only be free from the danger of bending, but also serve to support a temporary cofferdam of plank in case it is desired to unwater the culvert. This method of design, however, is not customary, and is, moreover, expensive.

Where culverts of the type shown in Fig. 190 are used for emptying, screens are occasionally provided for their entrances also. It is doubtful, however, whether this is necessary, and it is customary not to use them, as the upper screens generally furnish a sufficient safeguard. With rivers carrying very little drift, especially if the valves are of the roller or lifting type described on p. 498, the screens can usually be omitted.

No screens are provided for valves in lock gates, nor have they been found necessary.

The entrances to culverts should be enlarged to compensate for the reduction of area caused by the screens, and the upper culvert valves should be placed above lower pool level, so that access can be obtained to them without having to pump out the chamber.

Cover plates for recesses for machinery, etc., on the coping should be well fastened down if floods rise over the walls. An unusual flood on the Monongahela River has been known to carry away almost every cover plate on the series of locks.

The lower end of the lock, if not on rock, must be well protected against undermining from the turbulent discharge of the culverts or valves, as described on p. 414.

Manholes were provided in many of the older locks, opening vertically from the coping into the culverts. In the later designs, however, they have been omitted, as they have not been found of practical use, since repairs to the valves usually involve the removal of the water from the culverts, and access can be had through the main entrances.

Power for Operating Valves. (See also p. 485.)—Electric motors for operating counterweighted gate valves or butterfly valves, are usually of small power. Those operating the Stoney valves in the wall culverts of the Soulanges Canal locks in Canada under a 24-foot head are of 1 horse-power, with a starting effort of 3 horse-power. At the Horin lock on the Moldau the sector or Taintor emptying valves are operated under a 30-foot head by a 4 horse-power motor, and the horizontally-rolling filling valves by a 2 horse-power motor. The butterfly valves at the large Poses lock on the lower Seine are operated under a 14-foot head by a 3-horse-power motor.

Means for operating by hand power should always be provided in addition to machine power, for use when the latter gets out of order. Some of the sizes of gearing used in examples from actual practice is shown on Pls. 51 and 52.

It is customary to provide only one turbine or engine for operation, even when there are two or more locks, and experience has shown that one machine is all that is necessary, as in case of temporary breakdowns hand power can be resorted to. The sizes used in a few examples have been given on p. 485. It should be noted that when the gates are being moved no power is being used for the valves, and hence it is necessary only to supply power for the larger unit. It frequently happens, however, where

power capstans are used, that one or more capstans may be in operation at the same time as a pair of gates or valves, and provision must be made accordingly.

Wear of Culverts.—Where practicable the main culverts should have easy changes of direction, in order to avoid any tendency toward unequal wearing. Experience has shown, however, that the wear of the masonry is in general imperceptible, so that where economy is to be specially considered, there is usually no practical objection to introducing sharp turns or drops, as shown in Figs. 188 and 189, when accompanied by moderate lifts, up to say 15 feet. With high lifts, however, these should be avoided as far as practicable, or abrasion will ultimately develop. Where the changes in direction are not sharp, the culverts appear able to stand a high velocity without wearing. Thus in the outlet tunnels of some of the Niagara Falls power plants, lined with vitrified brick, no wear was found to have taken place after 15 years' almost continuous use under a velocity of 26 to 30 feet per second. The brick in fact was found to have become covered with a thick growth of weed and slime. In the culverts of the 32-foot lift lock on the St. Denis Canal in Paris, built of limestone and using about 150,000 cu.ft. of water per lockage, no wear was found after 12 years' use with several thousand lockages per annum. Similarly certain of the sluiceways of the Assuan Dam, lined only with granite and discharging (partially opened) under a head of 46 feet for several months each year, showed no appreciable sign of wear after 5 years' service. Sluiceways through Dam No. 11, Kentucky River, placed in the concrete and without any lining, showed no wear after several months' continuous flow under an 18-foot-head, and were found on subsequent examination to be largely coated with water weed.

The danger of wear in the culverts appears to be slight, as the extreme velocity lasts only a minute or two, owing to the rapid reduction of the head, and does not obtain more than a few times per hour. For this reason and in view of the experiences just quoted no protection appears to be necessary for the masonry.

Filling and Discharge Areas.—To determine the area required (which is usually made the same for the filling as for the emptying valves or culverts) the following rule may be used:

Let s be the horizontal area of the chamber;

A, the total net area of the valves;

h, the lift between pools;

g, 32.2 feet, the acceleration of gravity;

t, the time of filling or emptying in seconds, all valves being opened fully and simultaneously;

m, the coefficient of contraction, equal to about 0.62.

$$\text{Then}\quad A=\frac{s}{mt}\sqrt{\frac{2h}{g}}=\frac{s}{t\times 0.62}\cdot\frac{\sqrt{h}}{4.01}.$$

The time of filling is usually based on a rise of the water in the chamber of 1 to 4 feet per minute, in order to prevent too great currents with the attendant danger to craft.

Foreign practice favors the lower rate, but in the United States the higher rate is in common and successful use. The question depends chiefly upon the type of valve or culvert adopted and the size of the lock, as described in preceding paragraphs. At the large lock at Suresnes near Paris, with a lift of 10.7 feet and designed for a least navigable depth of 10½ feet, the rate of filling is about 1 foot per minute. At the Horin lock on the Moldau, with a 30-ft. lift and providing for a least navigable depth of 6½ feet, the rate is about 2 feet per minute. At the Poe lock of the St. Mary's Falls Canal, Michigan, with chamber dimensions of 800 feet by 100 feet, filled through the floor and handling the largest ships of the Great Lakes, the rate is about 2 feet per minute, the lift being about 20 feet, and at the 24-ft. lift lock of the Soulanges Canal in Canada (filled through side culverts as in Fig. 191, p. 492), handling ships up to between 2000 and 3000 tons, the rate is about 4 feet per minute. In the last-named case no trouble appears to be experienced from the movement of the water, and tugboats sometimes lie against the chamber wall without troubling to tie up. For the locks of the Panama Canal, filled through the floor and with lifts from about 28 feet to about 30 feet, a rate of 3 feet per minute was allowed, and the new lock at St. Mary's Falls Canal with chamber dimensions of 80 feet by 1315 feet and a lift of about 20 feet is designed to be filled at the rate of 2.2 feet per minute. This lock is also filled through the floor, six culverts each 9 by 10 feet being used, extending along the lock from the head to the tail bay.

In a comparison of a number of locks in the United States we found that the proportion of net opening for filling or emptying the chambers varied from 1 square foot in 1800 cubic feet to 1 square foot in 4000 cubic feet, the first ratio applying to locks of low or moderate lift (up to 9 or 10 feet), and the second to locks of high lift (up to 18 feet). The cubic feet referred to are based on the number of cubic feet of water required for a lockage when both pools are at the crests of their respective dams, and the ratio is therefore independent of the size of the chamber. In certain of the large locks of the lower Seine the proportion is 1 in 4300, and in examples on the Moldau 1 in 3400. In the Poe lock at St. Mary's Falls the proportion is 1 to 4200; in the new lock at the same locality it is 1 to 3900; and in the Ymuiden lock of the Amsterdam Ship Canal (completed about 1900) it is 1 to 3300 under a maximum lift of 5 to 7 feet. This lock has a culvert along each wall, with cross openings into the chamber, as on the Soulanges Canal.

For the river and canal locks of the Barge Canals of New York State, 57 in number, and which are 45 feet wide and 310 feet in available length, with 12 feet least depth on miter-sills and lifts from about 6 to 40½ feet, the following sizes of culverts were adopted in 1906. For lifts up to 13 feet, 5 by 7 feet; for lifts from 13 to 22 feet, 6 by 8 feet; for lifts above 22 feet, 7 by 9 feet. One culvert was placed in each wall, the water entering through the upper gate recess, and passing behind the lock gates into or out of the chamber through small openings (usually ten to each culvert) as shown

in Fig. 191, p. 492. The culverts ran the whole length of the wall and emptied into the tail bay. Their ratio of net opening to cubic feet of lockage, as referred to in the preceding paragraph, was about 1 to 1500 for an 8-ft. lift; 1 to 2800 for a 13-ft. lift; 1 to 2200 for a 14-ft. lift; 1 to 3500 for a 22-ft. lift; 1 to 2800 for a 23-ft. lift; and 1 to 4600 for a 38-ft. lift. The sizes were chosen on the basis of an allowance of about 5 minutes to fill or empty the chambers, except with the higher lifts, where the movement of so large a body of water in that limit of time might be attended with too much violence of current. The entrances to the upper culverts were widened so as tc avoid any choking of the inflow.

For the locks of the new Welland Ship Canal (Canada), each 80 feet wide by 800 feet in available length with a lift of about 46½ feet, the culvert area is 1 square foot to each 7500 cubic feet of lockage, and the rate of filling was assumed as 5.8 feet rise per minute, with a total filling time of eight minutes. The culverts run along each wall and open into the chamber through numerous side ports, as with the locks of the New York State Barge Canal. When filling high-lift locks of this character it is usually desirable to open the valves only part way for a minute or two, in order to avoid an undue disturbance in the chamber.

In selecting the area for culverts or valves, it is well to make it amply large. If too small delays in locking will ensue, and there will be no remedy. If on the other hand it is found that the area is somewhat large, causing disturbance when operating under full head, the flow can always be checked by partly closing the valves. The advantage of the excess area in such cases comes into play as the head gradually lessens, and the filling and emptying can then be expedited by opening the valves fully. In addition to this, if an accident occurs to one valve, putting it out of operation, the delay in lockage will be reduced if the remaining valve area is ample.

CHAPTER IV.

FIXED DAMS.

General.—While the construction of fixed dams has been practically abandoned on the Continent for navigation purposes, they are frequently employed in America, where conditions in general are better adapted to their use than is the case in Europe. They have been built, and are still being built, where the conditions are also favorable for the construction and operation of movable dams, because they can be constructed of cheap materials, usually abundant in the locality, and because they require but little attention for some years after completion, except occasional repairs.

In streams having small commerce or with high banks the use of fixed dams is not usually objectionable, while they are well adapted for purposes of furnishing power. To the latter fact was partly due their use in this country, because nearly all the earlier river improvements were made by corporations or by State governments, and one of the chief objects was to secure power for industries as well as water upon which to transport their products. These streams have since come under the control of the United States, and with the advent of steam and the creation of industries elsewhere the use of power from their pools was greatly curtailed or entirely abandoned. The value of the cheap power, however, is again becoming appreciated; and in some cases the improvement has been designed with a view to its utilization. It has been found necessary to rebuild many of the older dams where made of wooden cribs filled with stone, a class of construction which the attacks of floods and drift and the alternate exposure to air and water injure sooner or later, ultimately necessitating extensive repairs.

Alignment and Length.—Fixed dams are generally placed near the head of the lock, this location being adopted in order to avoid creating strong currents in the lower approach. They are usually built straight and at right angles to the axis of the river, but other forms may be seen in old structures, especially in Europe, sometimes with a curve of large radius, sometimes with straight or broken arms inclined more or less to the axis of the river, in order to obtain a longer spillway. It is important that a dam of the fixed type should restrict the waterway as little as possible, and therefore it ought to be as long as the conditions will permit. The greater the length over which the action of the water is extended, the less will be its tendency toward undermining the works and the banks, and the greater will be the facilities for discharging floods.*

* Experiments by M. Mary with a rectangular sluiceway discharging about 5 second-feet showed that with the weir at right angles to the sides the depth on the crest was about 0.18 ft.; when placed at an angle of 1 to 1 the depth became about 0.07 ft.; at 1 across and 2 upstream, about 0.06 ft; and at 1 across and 3 upstream about 0.03 ft. These relative depths were about 100, 40, 30, and 20 respectively. In practice, however, a dam on a river where flood levels vary considerably usually becomes "drowned" above certain stages, so that is it only when there is a fall that an inclination of crest line would affect the discharge as just described.

No rules of much practical value can be formulated for determining the length of a fixed dam, as can be done for movable dams. In theory it should be such that just before the lock is drowned, boats can pass over the dam in either direction; in other words navigation must never be interrupted. (See also p. 394.) Practically, however, this condition is affected by a number of external elements whose combined effects cannot be foretold, such as the rapidity of the rises, the length of the pool below, the width or narrowness of the river in the first mile below the dam, and similar conditions. The only safe rule is to follow that of experience, which shows that the spillway should be as long as practicable. A dam that is too short is liable to produce a dangerous scour below it, which may threaten or even undermine some of the construction (see Chapter X), besides creating currents and eddies during floods which will hamper navigation. This matter is referred to further on pages 534 and 535.

General Design and Construction of Timber Crib Dams.*—In the United States a great number of stationary dams have been built of timber cribs filled with stone, and decked or floored over. This deck is sometimes made a continuous slope from the crest to near the lower pool level, and sometimes is broken into steps of varying heights and widths. The former are known as slope dams, the latter as step dams. (See Pls. 54, 55 and 56.) Where practicable these cribs should be founded, at least along the face or downstream side, upon solid rock. The slope and the step dam have each their advantages and drawbacks. The decking of the latter is more easily injured by the passage of ice, drift, saw-logs, etc., but in moderate stages it retards the progress of the water so that it arrives at the lower pool with less velocity than if it had passed over a slope, and hence its effect is not felt so far below. It is also a little cheaper to construct than a slope dam. Neither type appears to have a decided advantage over the other, some engineers preferring the slope, and others the step design. In some cases, as on the Kentucky River, old step dams have been repaired by covering the steps with 3 to 4 feet of concrete laid on a slope, thus changing the structure to a slope dam.

The bottom width of the dam must of course be sufficient to prevent overturning, and its foundation strong enough to resist the pressure that is to come upon it. The ordinary timber and broken-stone dam is usually much wider than its height, one reason for this excess of width being that it gives a more gradual descent to the water from pool to pool, and thus reduces its undermining effect. In one example of old construction, with a lift of 17 feet and a rock foundation, the base width is 43 feet. Another, recently built, with a similar foundation, has a lift of 18 feet and a base of 50 feet, and is 30 feet high. A third has a lift of 14 feet and a base of 33 feet.

* Tables of lifts, etc., of various dams will be found at the end of the book, and Pl. 46 shows the location of the principal canalized rivers of the United States.

The construction of timber dams has been abandoned in the majority of cases in favor of concrete dams since the original publication of this treatise, but it is believed that the description of the methods used may prove of sufficient interest to justify its retention herein.

The natural foundation will largely govern the design of the dam. If of rock, the case will not present any special difficulty, but if of gravel or other light material, great care must be used to prevent washing and undermining. In cases where the river is deep the cribs usually rest directly on the bottom, but where it is shallow a trench is dredged out and the cribs are sunk in it, or piles are driven and cut off just below water and the crib-work built upon them. If the last method is used, an apron crib should always be sunk against the downstream side of the dam as deep as possible, or the reaction may undermine it. Experience has shown that aprons formed simply of piles framed and decked over, or of cribs resting on piles, are unsuited to rivers of high floods, as the boiling of the current is very apt to tear them loose, and in more than one case serious damage has resulted from their use. If such a design must be used, the parts should be well bolted together and be anchored to the river bed. It should always be remembered that the vulnerable part of a fixed dam or of a weir is its downstream side, and as much care must be taken in protecting it as in securing the upstream side.

Where the dam is to rest upon a pile foundation, in whole or in part, a pile is provided at each of the intersections of the crib timbers, and one row close together along the downstream face, unless an apron crib is used.

Details of Construction.—The crib dam of modern design is usually built of sawn timbers, 10 inches to 12 inches square, laid crosswise so as to form pens 8 to 12 feet centers, and filled with riprap. The stone is usually of small sizes and irregular shapes called "one-man stone," but sometimes these are taken out in large blocks as blasted in the quarry, and placed with a derrick. Limestone and sandstone are both utilized, but the latter wears rapidly through the action of water in dams which have leaks through the upper face or spaces in decking. The cost or inaccessibility of stone may sometimes render it necessary to use gravel for filling the cribs, but this usually contains so many small particles that will be washed away that its use is not desirable. In such cases the downstream side of the dam should be planked to hold in the filling.

The timber generally used is white oak or yellow pine, but any of the heavier woods may be employed, if under the low-water line, or where they will always be moist. It may be either sawn, hewn, or left in its natural state so far as the cribs themselves are concerned, but the deck or floor covering the whole structure should be either sawn or hewn. At the present time hewn and round timbers are very rarely used, as sawn timbers can usually be obtained for a little extra price, and are much preferable for rapidity of construction. The pieces may be laid directly upon each other, or daps may be cut at their intersections, at which points they are drift-bolted. The former method, however, has proved as satisfactory and as durable in practice as the latter, and is less expensive.

The timbers which lie in the direction across the stream are called stringers; those

which have their direction parallel to the current are called ties. Both sets of timbers, especially the ties, should have a length as great as practicable, economy being considered, and the joints may either butt or be spliced. If the former, a block of similar section to the stringer or ties should support the joint and lap onto each timber about 2 feet, drift-bolts being used to connect the block with the stick below as well as with the one above. (See Pl. 55.) These joints should not appear immediately over each other in two succeeding layers of timber. If the joint is lap-spliced, it may come at the intersection of the timbers with those lying in the opposite direction; if it does not, then it should also be supported by a block, although this is rarely done. Butt joints, however, have proved quite satisfactory in practice and are now used almost entirely. Stringers inside the body of the dam need not have the ends sawed close, nor have their joints come over a tie. The drift-bolts used are usually $\frac{5}{8}$ or $\frac{3}{4}$ inch square, depending on the size of the timbers, with heads and with wedge points. Round or nail points should not be used, as they split the timbers, and square drift-bolts are preferable to round ones, as they do not turn in driving. Such turning with wedge points is apt to split the lower timber, if driven near its end. The top timbers are bored with augers of the same size as or slightly larger than the bolts (depending upon the liability to splitting of the kind of wood employed), but the bolts are driven directly into the under sticks, without boring the latter.

Both the upstream and downstream sides, known as the back and face of the dam respectively, are carried up vertically, the former being covered with a single or double row of sheet-piling, driven as deep as possible; this should extend to the top or near the top of the cribwork, and there connect with the decking of the upper slope. Some engineers cut it off about 2 feet below the top, and continue it with short plank, for easier repairing. Immediately upstream of this sheeting should be placed an embankment of gravel and clay, or of good earth, preferably riprapped on its upper surface for a distance of 10 or 15 feet away from the dam. This will tend to safeguard the dam by increasing the percolation or seepage distance, as described on p. 367. The downstream face may be left open between the timbers unless the spaces exceed 7 or 8 inches, when they should be reduced by fillers, or the filling stone will be washed out. The upstream face may be carried to within 3 or 4 feet of crest height; the lower face should stop at or near lower pool level.

The line of the crest is ordinarily placed from 8 to 12 feet from the upper face, and the sloping decking (which is placed as described for step-dams) connecting these points should be practically water-tight. It is placed on a slope to permit the passage of drift, etc.

If the dam is of the step type, the height of each step should not exceed one-third the width of the next below, or in certain stages the falling water will miss the step and strike the one below, and if drift or ice is running, the decking will suffer accordingly. Thus, where the steps are 10 feet wide, the limiting height should be

GENERAL VIEW OF DAM No. 7, KENTUCKY RIVER, KENTUCKY, U. S. A., DURING CONSTRUCTION.

The dam is of the step type and built of timber cribs filled with riprap. The length is 350 feet, with a base of 60 feet. The lift is $15\frac{1}{2}$ feet. The dam was completed in 1897.

GENERAL VIEW OF DAM No. 1, BARREN RIVER, KENTUCKY.

The dam is of the slope type with a comb-stick, and is built of timber cribs filled with riprap. The length is 268 feet, with a base width of 80 feet and a lift of 15.1 feet. The original structure was completed about 1840, but has been practically rebuilt several times since.

about 3 feet 4 inches. The step descending to the apron, however, if the latter is wide, may be as high as 5 feet. The width of these successive steps may vary between 8 and 12 feet, 10 feet being usual. The decking of the upstream slope should be of two thicknesses of 2-inch or 3-inch white oak or hardwood plank, placed so that when the top layer is worn it can be renewed without destroying the lower one. The decking of the other steps may be from 4 to 10 inches in thickness, depending largely upon the amount of ice, drift, etc., passing over the dam, which soon wear away the surfaces. On rivers with high floods a 10-inch thickness is customary.

In certain cases the crest is stopped from 6 to 12 inches below the upper pool, and the remaining height is obtained by spiking a "comb-stick" on the decking (see Pl. 55). This stick should be of oak, and be well secured with drift-bolts and straps, as it has to stand heavy blows from drift. It is frequently necessary to use this method in order to level up or raise the crest of a dam which has settled, which always happens as the timbers become old, and in point of fact the settlement usually dates from the completion of the work. No apprehension need arise, however, if settlement occurs while the cribs are being filled, as the weight of the stone usually produces such a result, especially towards the center of the river. In one crib dam about 20 feet high, on gravel, a maximum settlement occurred of about 10 inches along the crest, the apron crib, which was 12 feet high, remaining practically unchanged. Also, if the crest develops a bow downstream as the water pressure comes against the dam, no alarm need necessarily be felt, as such a movement is due to the lack of rigidity in the joints. With high cribs a bow of this nature almost always occurs, as it does in fact in any high cribwork where the pressures are not balanced. On one crib dam of 18 feet lift and 30 feet in height, with a total base width of 50 feet, a bow gradually developed in the crest of about 18 inches, nearly all of which had been produced by the time the new poo reached the crest. Practically no increase took place afterwards

With slope dams the upstream side is built as for step dams, and the downstream side is made with a slope varying from 2 feet 6 inches to 4 feet of base to 1 foot of rise. The first-named slope is somewhat too steep and is apt to cause a violent reaction below the dam in high floods. A slope of $3\frac{1}{2}$ to 1 is much better, and some engineers prefer 4 to 1 where floods run high. As generally built these dams provide for but little protection below, but it has been found necessary in almost every case to place apron cribs below them later in order to prevent undermining. Where this method is adopted in the first case, as it always should be, the cross-section of the dam itself may be reduced, as the apron can be made part of the structure. Usually there is little need of protection below a dam with slopes of 4 to 1.

The apron is occasionally made with a considerable upward inclination in the downstream direction, this construction having been found advantageous in carrying the reaction away from the toe and in some cases causing deposit along the downstream face of the dam. (See p. 547 and after, "Foundations and Details of Construction.") The

decking should be placed a little above the lower pool, so that it can be easily inspected and repaired during low water.

The width of the apron crib, which in a step dam is formed by the lowest step, is dependent on the depth of water below it and on the lift, since the higher the lift the greater will be the width required. For a gravel foundation it may be determined on the proportion of 18 inches to 2 feet of width for each foot of lift, with a minimum of 8 or 10 feet. Thus for a dam of 4 feet lift the apron would be 8 or 10 feet wide, while for a dam of 12 feet lift it should be 18 feet or more. (See also formula on p. 375.) These figures are for rivers of high floods; where the floods are small the apron can be greatly reduced (see Pl. 56), while for high floods and sandy foundations its width must be correspondingly increased. After a flood has reached a depth of 4 or 5 feet on the crest, the profile of overflow becomes the same for slope as for step dams, and a violent reaction occurs where it strikes the lower pool, and not infrequently pieces of the apron decking are loosened by it and sometimes a large number torn up at once. Damage is frequently done also by blows from drift, which at certain stages may be caught in the reaction and be held floating just below the dams for several days. Especial care should therefore be taken to fasten the top stringers to the stringers below, and it is best to make the former of oak or hardwood, which will hold fast the drift-bolts of the decking. If these precautions are taken, pieces will rarely come loose. The decking of the steps or slope above seems to remain practically unaffected by the movement of the water, although if there is no apron crib or if it is too narrow the ends of the decking timbers are often battered by the drift driving to and fro as just described.

In some cases concrete has been used for decking as described on p. 506, and has proved very satisfactory.

An instance of the value of an apron occurred at Dam No. 1 on the Kentucky River, which, after standing for many years as a timber step dam, was repaired and changed to a concrete-decked slope dam, the existing apron being included as a part of the slope. During the following winter the changed conditions caused a scour in the gravel bed to an extent which threatened the safety of the structure, and this only ceased after an apron had been added. This apron has a width of 20 feet and was built 2 feet higher at its downstream edge than at the junction with the dam proper, with the purpose of causing a filling in rather than a scour immediately below, and in this it has been wholly successful. Similarly at Dam No. 12 on the same river (generally similar in outline to Dam No. 14, shown on Pl. 57, and built on a shale rock without an apron) the reaction scoured out the somewhat soft bed-rock to a depth of 3 or 4 feet during the first winter's floods.

The deck timbers on the downstream side of a slope dam and on the steps of a step dam are sometimes spaced $\frac{1}{2}$ to 1 inch apart, in order to permit a freer movement of any entrained air and the quicker escape of the overflowing water. This plan, however, appears to be of little practical value, as it has no appreciable effect in facilitat-

ing the discharge, and it has been found in addition that it is liable to wear away rapidly the filling immediately underneath. With the apron decking, however, it is considered best to space the timbers a little apart, as it is believed to check the reaction in ordinary stages, and to reduce the tendency to loosen the timbers. Where the reaction has been violent, however, open-spaced decking has often been torn off, notwithstanding the fact that the pieces had been placed an inch or more apart.

Methods of Construction—Without Cofferdam.—Crib dams are usually constructed during the low-water season and without the use of a cofferdam. This latter feature constitutes one of their chief advantages, as it permits of cheap and rapid construction. The site is first cleared, either by dredging or otherwise, of all obstacles which might interfere with the proper seating of the cribs, and then is carefully sounded throughout in order to determine the location of any irregularities. The cribwork is then divided into sections of such dimensions as it is considered can be handled in the current with the appliances available. These may be up to 100 feet in length and of the whole width of dam, and in practice they are generally put together immediately over the place where it is proposed to sink them; but this may be done, or partly done, elsewhere, and they are then towed into position when ready. The profile of the bed, having been determined by the soundings, the under side of the crib is shaped approximately to fit it by means of additional timbers or blocks, drift-bolted on where required, so that when the crib reaches the bottom its top will be nearly level. The cribs are built in the water, sinking as timber is added until the bed is reached. They should then be weighted with stone on top to settle them. The construction may begin at one or both sides of the river. Only sufficient stone is immediately placed to hold the crib in position, as it is desirable to allow the water to pass through with as great freedom as possible until the dam has been nearly completed. As soon as one crib is built up to a height somewhat above the water stage, and is safely anchored, another is sunk at its outer end until all are in place. The discharge area of the river will have been contracted considerably by the time the last crib is put in, and the velocity increased, so that it may be necessary to carry anchorages or wire ropes upstream to prevent displacement of the cribwork. Should slight displacement occur, however, it is usually not of serious import, as the timbers above water-level can all be carried through on the proper line. If the dam is on rock, with therefore no danger of the river cutting under the cribs, the work of connecting them by a continuous crib covering the several joints should be vigorously carried forward. During this time there should be on hand an abundant quantity of stone which can upon short notice be dumped into the cribs in case of an approaching rise, and some of it should be put in place as the work progresses. When the cribs have reached their full height the filling should be rapidly completed, and after that the decking may be placed. Finally, after all has been done and satisfactory connection made with the lock wall and abutment at the ends, the sheet-piling along the upper face must be put in place. This should

invariably be done with great care and should extend to the rock whenever practicable. As this approaches completion the water above will rise and flow through the valves in the lock, and if the discharge of the river exceeds the capacity of these valves it will eventually rise over the crest of the dam. It is, therefore, necessary to proceed rapidly with the sheet-piling, and this portion of the work should be carried on continuously and at several points. Where there is a considerable flow of water it may be necessary to open the lock gates and depend upon the erection of the cofferdam in the head of lock (see p. 419) to close them again with safety. This is not advisable, however, unless the conditions for continued dry weather are favorable, as a sudden rise might render it impracticable to place the cofferdam in position, and the increased flow passing through the lock might endanger the safety of the gates or even of the structure. When the sheet-piling has been completed an embankment of gravel and earth or of good earth only should be placed immediately above, as has been described on p. 508.

Where the foundation is of light material, so that there would be danger of undermining before the sheet-piling could be put in as just described, the latter should be driven as soon as the foundation is above water, and gravel and brush should be kept on hand to stop any under-cutting. In this case the piling is sawed off at the water surface and the river flows over it, and when the dam is finished the spaces above are closed by a double row of planks resting on the sheet-piles and spiked to the crib timbers. This method is usually to be preferred in fact in all cases where the dam does not rest on bed-rock, as the sheet-piling can be more easily driven, and there is less danger of trouble from the increasing head of water.

With Cofferdam.—There are cases, as in the construction of masonry dams and occasionally of timber dams, in which it is necessary to perform the work inside a cofferdam, or rather inside two or more cofferdams, since it is not practicable to close the whole opening at one time in a stream of good discharge. The type of cofferdam to be used will depend upon the character of foundation upon which it is to be built and the style of dam proposed. The different kinds in use and the methods of construction have been described on p. 348 and after.

If desired, the cofferdam may be designed so as to become a part of the completed structure, or it may be left to form a protection, the downstream arm being reduced to the proper height and floored over for this purpose. The arms of the cofferdam are carried out from either the abutment or the lock, but generally from the side having the poorer foundation, so as to throw the increased current caused by the contraction over the part of the foundation better able to resist erosion. When the dam has been completed as far as desired within the inclosure, a bulkhead or cross cofferdam reaching several feet above future crest level is built near its outer end, connecting the two cofferarms. This done, the latter may be removed, and the cofferdam for the next portion of the dam may be put in and be pumped out, and the work proceeded with.

The discharge of the river during the latter part of the construction may be passed through the lock, or through sluices left in the dam, or it may flow over unfinished portions of the dam, these portions being later on built up alternately a foot or two at a time, using needles or stop-planks to keep the water off the masonry being placed. Dam No. 2 on the Allegheny River, considerably over 1000 feet in length, was built in sections between 20 and 30 feet long to its full height, each alternate section being at first omitted so as to pass the flow. The final closing was made during low water, all the dam having then been completed except a few sections. These were closed by timbers on the upstream and downstream sides and were then at once filled with concrete. This method is in general the one to be preferred, although sluiceways temporarily left open through the masonry have been used very successfully.

General Design and Construction of Masonry Dams—(Concrete and Stone).*—(See Pls. 54, 57 and 57*a* and Figs. 198 to 201; also p. 360 and after.) With the increase in the cost of timber, masonry dams have become more widely used, and in many cases, especially with rock foundations, their first cost may prove no greater than that of crib dams, while for watertightness and economy in maintenance they are greatly superior.

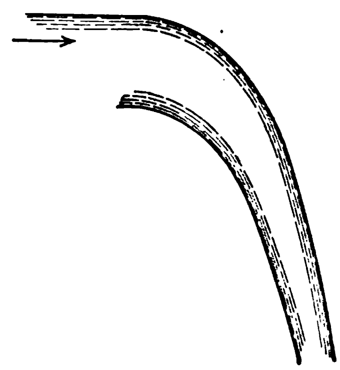


FIG. 194.

The general outline of discharge over a crest into free air is shown by Fig. 194; the falling water takes an approximately parabolic outline, varying according to the velocity of approach and the depth of fall. The greater the velocity the further the sheet is projected beyond the crest, and the greater the fall the more nearly does the lower portion approach a vertical line. The shape of the crest also has a considerable influence on the discharge, a broad flat crest passing a smaller given volume of flow than a rounded or sloping one. The downstream slope or "face" should also be flat enough to prevent the water from discharging into free air, and thus creating a tendency to form a vacuum. A broad flat crest tends to reduce the discharge, as the water, except in high stages, takes an outline about as shown by the full lines in Fig. 195; the broken lines show the same discharge with a rounded crest. On the other hand the flat crest possesses the sometimes valuable feature of passing ice with comparatively little injury to the structure, as described on p. 533 under the clause "Ice and Fixed Dams." The upstream edge of the crest should in all cases be beveled or rounded to prevent injury from ice or drift and to ease the approach of the water.

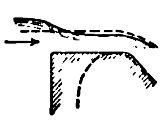


FIG. 195.

Where the downstream side is very steep, as illustrated in Fig. 196, the natural path of the water is about as shown by Fig. 194, but as it strikes the lower pool it sucks down

* See p. 430 and after for remarks on building concrete structures.

part of the air in the space ABC and creates a partial vacuum. Under normal conditions the pressure of the atmosphere on the upper-water surface, as shown by the arrow at D, is offset by a corresponding pressure on the face as shown at E. When therefore this latter pressure is removed or partly removed, that at D becomes unbalanced, as noted further on, and produces the same effect as an added head of water. When the water is running over the dam, the weight of the atmosphere on the outside presses the falling sheet inwards, and the contest of the opposing forces—the water trying to keep its natural path and the atmosphere trying to obliterate the vacuum beneath it—produces a vibration of the falling sheet often sufficient to disturb the air for a considerable distance, and sometimes causing a rattling of house-windows in the neighborhood. The latter effect is often annoying, but the "tremblings," as they are frequently called, can be stopped by spiking pieces on the crest or by taking other means to admit air under the falling sheet. The greater the depth of water passing over the crest, the less are the vibrations, as the body of water becomes too large to be easily affected, but they have been observed up to depths of 2 or 3 feet. The effect of the vacuum is of course just as active in these cases as when the discharge is small, and probably more so, as the exhausting power increases with the volume.

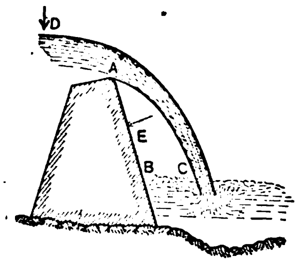


FIG. 196.

The precise effects of the tendency towards a vacuum or partial vacuum caused by a too steep face cannot be told, as there are often local conditions of flow or of construction which allow air to be drawn under the falling sheet and thus prevent any detrimental results. Moreover, if the space between AB and the natural outline of the discharge AC is very small (Fig. 196), the effect of the vacuum is correspondingly small, as the outside pressure of the atmosphere tends to force the surface AC against AB, and when contact is secured no additional vacuum can occur. If it were possible for a perfect vacuum to exist, however, the theoretical overturning pressure on the dam would be increased by about 15 pounds per square inch (the natural pressure of the atmosphere) over all the surface thus affected. As this would be equivalent to the pressure of an additional head of about 35 feet of water on each square foot of vacuum surface, it will be seen that the effect might easily prove disastrous. In point of fact, several failures have occurred of dams apparently strong enough to support their static head, but whose downstream slope was steep enough to afford space for a partial vacuum. On the other hand, dams with similarly steep slopes are in successful use, but in these cases there may be local conditions present which help to destroy the vacuum, as alluded to above. Whatever unknown causes may be at work, the presence of strong vibrations in the falling sheet of water, often seen with dams of steep or vertical face, shows that a vacuum is present, and experience appears to indicate that it is wise to guard against the possibility of dangerous effects. A radius of 4 to 6 feet is usually large

enough to pass the water over the crest without its leaping clear, except for rivers with high stages and unusual velocities, when the radius should be increased. This radius can end in a slope of $1\frac{1}{2}$ vertical to 1 horizontal for dams of moderate lift, and if the lift is high the slope can be made steeper, up to a limit of 2 vertical to 1 horizontal. In these cases the crest radius is usually increased to 10 or 12 feet and sometimes more. The slope should end in a curve of a radius of about 10 to 20 feet (depending on the lift and on the height of floods), which in turn should end in an apron of length suitable to the natural foundation. (See p. 511 and Pl. 57.)* Where a large radius for the crest is desirable, but its use would lead to an uneconomical amount of masonry if the profile of the dam followed customary outlines, the crest can be made to project as was done with the spillway of the Halligan Reservoir in Colorado. (Fig. 196*a*.)†

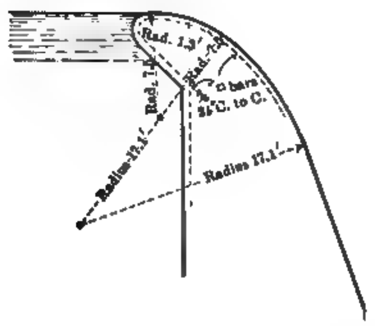


FIG. 196*a*. FIG. 197.—Cross-section of the Narora Weir, River Ganges. Head, 12 Feet.

The presence of underpressure or uplift with foundations of porous material is a matter about which actual knowledge is meagre. It is known, however, that the underpressure is gradually reduced as the seepage passes through the natural foundation on its way to escape below, as has been described on p. 365 and after. Fig. 197 shows an experiment made about 1897 at a dam in India on a foundation of fine sand, and with a deep cut-off wall along the downstream face tending to prevent the escape of any seepage. Two holes were drilled in the floor as shown and fitted with pipes, and the water rose in one hole to 6 feet above the lower pool level and in the other to 5.16 feet. Under the existing 12-ft. head, therefore, it appeared that the pressure was reduced 50 per cent in a distance of some 30 or 40 feet from the upstream edge of the puddle backing and about 8 per cent more in a further distance of 21 feet. It may be noted that this amounted to about 11 feet of head under the floor as against a thickness of 5 feet of masonry, the pressures thus being almost balanced. Later on a large

* For a full discussion of overflow see "Weir Experiments, Coefficients and Formulas," by Robert E. Horton, U. S. Geological Survey Paper No. 200, 1907.

† Transactions, Am. Soc. C. E., 1911.

section of floor, 900 feet from where the experiment was made, was forced up by the water, although the dam had up to that time stood successfully for several years.*

The general conditions to be met in the design, as underpressure, limits of resultant, etc., have been described on p. 361 and after. It is as necessary to investigate the stability during high water as during low water, especially with dams of high lift. The peculiar reaction which sometimes takes place during intermediate stages may also produce an increase of head, as shown in Fig. 197*a*.

Recesses for flash-board pins should be provided on the crest, 8 feet or so apart, to facilitate repairs or examination of the structure. They can be made of pieces of 2-inch gas-pipe, set about 9 inches into the masonry and flush with its top.

While a masonry dam is usually built solid there are several hollow types in use, some of reinforced concrete and others of massive walls divided into pens. On the Salt River in Arizona is an example of a concrete dam built on piers. The latter are spaced about 20 feet centers across the river and are about 9 feet high, and are bolted to the rock foundation. They support the superstructure, which is an ogee dam, rising about 26 feet above the piers. These types are best adapted to rock foundations, and for dams of high lift where a flat downstream slope would involve an excess of masonry they are often economical. (See also p. 407, "Reinforced Concrete Walls.") As far as experience has gone, those in use, where properly designed, appear to have been satisfactory, although it is doubtful whether they would support the hammering from drift such as takes place along the apron at certain stages of rivers with high floods.†

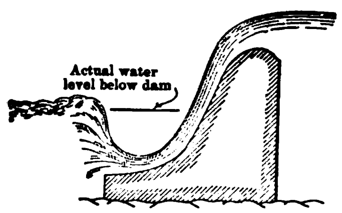


FIG. 197*a*.

The use of cofferdams and methods of closing a dam have been described on p. 514.

Ice Pressure on Dams.—It is not customary to make any allowance for ice pressure, either from field ice or from gorges, in designing dams, as mentioned on p. 372.

Foundations.—(See also p. 372 and after, and "Foundations," p. 547 and after.) While rock should always be selected for the foundation where practicable, masonry dams in rivers have been successfully built on all classes of material. The head supported has rarely exceeded 12 or 15 feet, so that the risks incurred do not compare with those of reservoir dams. As with movable dams, the chief dangers lie in undermining from above and from below, and good sheet-piling and backing can guard against the former, and a good apron and riprap against the latter. Figs. 198 to 201 show sections of dams in India, Egypt and elsewhere, some of them built on fine sand, and of a type which has successfully stood half a century's test. An example very similar to the Nile dams is to be found at Yuma near the mouth of the Colorado River, close to the

* Proceedings Inst. C. E., vol. clviii, 1903.

† The special features of certain types are protected by patents. We have been informed by an engineer who had designed and built many dams of reinforced concrete that the type showed no saving of cost as compared with solid dams unless the height exceeded 15 or 20 feet.

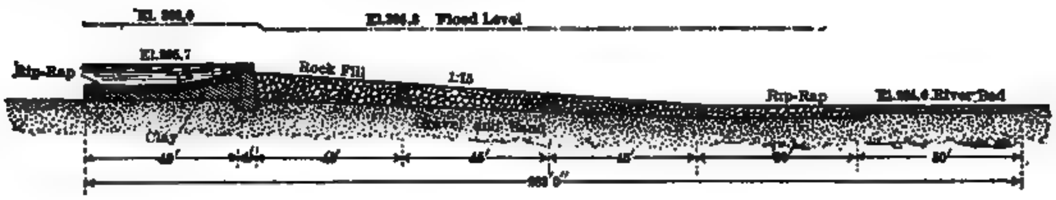


FIG. 198.—Madaya Weir across the Madaya River, Burma.

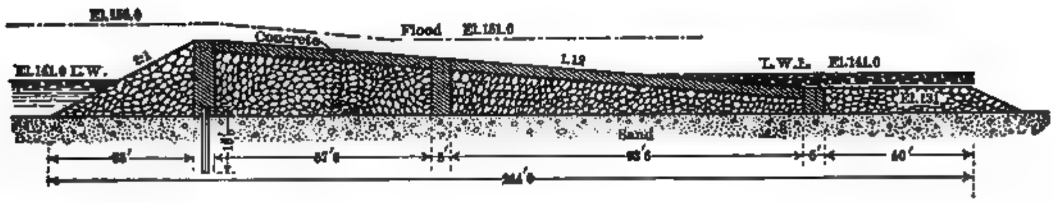


FIG. 198a.—Laguna Weir across the Colorado River, U. S. A. Length about 4000 Feet.

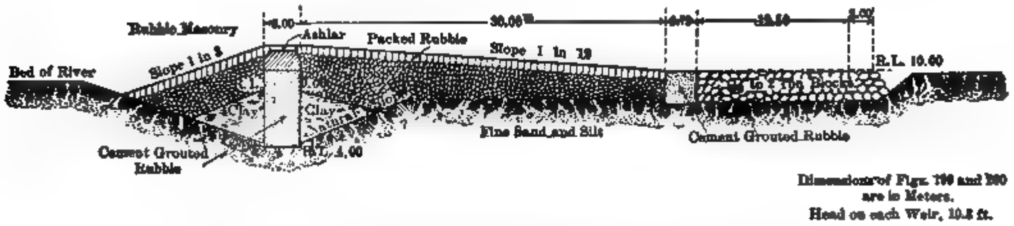


FIG. 199.—Damietta Weir, River Nile, 418 Meters Long. (1901.)

FIG. 200.—Rosetta Weir, River Nile, 500 Meters Long. (1901.)

FIG. 201.—Dehri Weir, River Sone (India). Lift, 10.0 feet. Length, 12,500 feet. Flood discharge, 830,000 Second-feet.

CROSS-SECTIONS OF FIXED DAMS ON SAND FOUNDATIONS.

(Figs. 198 and 198a, and Figs. 199 to 201 are reproduced by the courtesy of "Engineering News" and of the Institution of Civil Engineers respectively.)

border between Mexico and the United States. (Fig. 198*a*.) On some of these wells were used instead of sheet-piling. These have been employed a good deal in India. They are made of brick and built side by side, being sunk by excavation from the inside, and the spaces between made tight where practicable by grouting, and the wells filled with concrete. They are employed as being in many cases cheaper and more permanent than sheet-piling. The wells along the downstream faces are intended to prevent the seeping out of the material, but at the same time they tend to increase the upthrust on the foundation by confining the leakage from the upper pool.

With masonry dams on gravel and similar foundations it is often advisable to use piles, or to set the dam well into the natural bed, so as to minimize the possibilities of settlement from undermining, and sheet-piling and backing should be employed along the upstream face as described for crib dams (p. 508 and after).

Apron.—A suitable apron is usually more necessary with a masonry dam than with a crib dam, because the descent of the water is more sudden, and produces a more violent reaction. (See also p. 512 and "Foundations," p. 547 and after.) On gravel or similar foundations a stone-filled crib is sometimes used and appears to work satisfactorily, especially if a water cushion is always over it. (See Pl. 57.) The falling sheet should be led on to the apron by a curve and without sudden or violent change of direction (see p. 517). On rock foundations, if the lift is low and the rock hard, or if there is always an ample water cushion below, the apron can often be dispensed with until experience has shown its necessity. This was generally done with the earlier dams, some of which, however, eventually become undermined, while on others the rock has undergone much erosion and has had to be protected. On the Merrimac River, at Lawrence, Massachusetts, however, is a power dam built in 1847 and standing from 10 to 12 feet or more above its foundation. This is of hard volcanic rock, with no water cushion except in floods. The downstream face of the dam is almost vertical, and there is no apron, so that the flood waters, which sometimes rise more than 6 feet above the crest, plunge directly on to the unprotected rock. No trace of erosion, however, has been found, in spite of an exposure of more than 60 years. This dam sometimes produces vibrations sufficient to shake the windows of the adjoining town. (See p. 516.)

Where there is little or no water cushion below and the river carries heavy ice, the apron should be made heavy and be well reinforced with rods or bolted to the rock, or the ice may break pieces off, as has occurred on some of the power dams on the Hudson. The width of the apron may be proportioned for gravel and similar foundations by the rules given or referred to on p. 512, while for rock foundations it may be made narrower, as the undermining will be comparatively very slow. On the old fixed dams of the Meuse and the Moselle, of low lift, no aprons were provided in the original construction, but erosion soon endangered the works, and riprap aprons 40 feet wide were added later. In some cases these proved insufficient, with the result that some of the dams were washed out.

It has been found that a steep fall combined with a high lift and a comparatively narrow apron will produce a violent reaction below which will scour out the material close to the dam, while a less steep fall with a similar width of apron will lessen the scouring effect. Thus on one dam on a tributary of the Ohio River, of crib construction with an 18-ft. lift and steps 5 feet high and 10 feet wide, the reaction scoured out all material lying close to and near the dam to bed-rock (about 18 ft. below the lower pool), and made necessary special fastening of the apron timbers, while at the next dam below, of similar construction but with a 15-ft. lift and steps $3\frac{3}{4}$ feet high and 10 feet wide and the same width of apron the worst scour occurred a little way below the dam, and the reaction piled up the gravel against the apron.

Movable Crests.—It is often desirable to place a movable crest on a fixed dam, so as to give a higher pool in low water than the fixed part alone would afford. The advantages are several in number—the amount of masonry in the dam can be reduced; lower lock walls and lock gates are required; there is less overflow of land during floods and a higher pool level can often be used in consequence; and there is less violence of reaction and more moderate currents below the dam. Thus on the 18-ft. lift dams of the upper Kentucky River, referred to further on, the 6-ft. movable top practically reduces the effect of the overflow during a flood to that of a dam of a 12-ft. lift.

The simplest method is to place pins about 8 feet apart along the crest of the dam, and to lay 1-inch plank against them. The latter are called "flash-boards." The pins may be of wood, but are usually of iron bar or of gas-pipe. When a flood comes many of the boards break and relieve the flow, and when the water recedes the bent pins are straightened and the lost plank replaced from a skiff or by men walking on the crest. This plan works well up to depths of about 18 inches, but above that depth care must be used or the current will give trouble during the placing of the boards. At several power dams in New England, however, as at North Lawrence, Massachusetts, tiers of flash-boards, aggregating $4\frac{1}{2}$ to 5 feet in height, have been used for many years. Provision is made for two parallel rows of pins, one upstream of the other; the downstream row supports the boards and the other row is used as a guard for the operating boat and is usually removed when the boards are in position. The latter are placed from the boat, which works out from the shore and is secured thereto by a line. They are carried away by floods in the usual method, the top tier usually going some time before the lower tiers, owing to the bending of the pins.

A more complicated method, but one which is necessary where the dam is of any length and rises are frequent, or where several feet of water are to be sustained, is to use some type of movable dam. The drum wicket and the rolling dam, described in Chapter IX, have proved excellent for this purpose, while the needle dam and the gate dam (Chapters VI and VIII) have also a good record, though more cumbersome in operation owing to the number of pieces to be handled. In fact, almost any type of movable dam can be applied, though the so-called automatic ones, such as the drum, have the

advantage of simplicity in maneuvering, especially for rivers of any width, provided that they keep in good order. This, however, cannot be depended on, and hence the more cumbersome but more reliable non-automatic types are generally used instead. On a power dam at Taftville, near Norwich, Connecticut, there was applied a series of self-operating shutters each 6 feet long and about $3\frac{1}{2}$ feet high, balanced about one-third of the height and closely resembling Chanoine wickets, except that they remained always in place on fixed supports and did not lie down behind a sill. They were designed for a head of about 3 feet and cost about $10 per foot run. The hinges of the shutters were made eccentric in pattern, so that when the head of water began to move a shutter the center of support would begin to rise accordingly and would thus keep the area of discharge proportionate to the head. It was found in practice, however, that the shutters did not move until the head of water sufficed to throw them over horizontally, and then they moved suddenly like the Chanoine wicket. With this exception they are said to work satisfactorily, though they would be unsuited to many other American rivers, as the shutters and framing would be injured by drift. A description of the installation can be found in "Engineering News," June 12, 1902

Drum crests, giving an additional depth of water of 3 feet, have been used on Dams Nos. 2 and 3, Monongahela River, the one at Dam No. 2 having been completed in 1906 and the other in 1908. The lengths of the sections vary from 40 to 49 feet each. Additional details will be found in Chapter IX.

Dam No. 5 on the same river, completed in 1910, has a movable crest composed of steel and wood shutters 3 feet 8 inches high and in lengths of 12 feet, giving a depth on the sill of 3 feet. The design is similar to that on the Betwa dam in India, where a crest length of 3600 feet is closed by 300 shutters, each about 12 feet in length and 6 feet in height.* The shutters are loose except for hinged tie bars on the upstream side, running from near their centers to the masonry. When the water rises a few feet above the tops it forces the shutters down, the bottoms sliding upstream along the masonry, and the shutters turning on the hinges of the tie-bars. Those at Dam No. 5 are raised from a boat as the water falls, the boat lying against the shutter first raised and working outwards.

Crests of Boulé gates (see Chapter VIII) are used on the Rocky Rift dam of the Mohawk River, N. Y. (1911), and on Dam No. 11 of the Muskingum River, Ohio. The former has a depth of about 3 feet on the sill with trestles 4-ft. centers and the latter has 4 feet on the sill with trestles 5-ft. centers. The trestles of the Rocky Rift dam are operated by hand from the walk-way by separate chains attached to the trestles, as the current and the fall over the crest are slight during floods, while those of Dam No. 11 are operated by a winch on the lock wall. Two tiers of wooden gates are used at Dam No. 11, each 2 feet high, the upper tier being $2\frac{1}{4}$ and the lower tier $1\frac{3}{4}$ inches thick. They are stored on a boat when not in use. The top tier is removed

* "Engineering News," June 4, 1903.

by hand and the bottom one by lowering the forward trestle; this removes the support from the outer end of the gate, which is then carried out by the current, having first been attached to a line by a snap-hook.

Dams 11 to 14, on the Kentucky River, which have a low-water lift of 18 feet, obtain 6 feet of this amount by movable needle crests (Fig. 201*a* and Pl. 57).* The trestles are spaced 8-ft. centers, permitting each trestle to be lowered independently of the next. This is of much importance, since trestles which cannot be thus maneuvered are liable to be caught by drift if a mishap prevents one of them from being lowered. The trestles were operated at first by separate chains attached to the heads, but the swift current over the masonry crest and the drift soon chafed and broke them, and connecting rods, tried later, fared no better. The trestles in consequence are now operated by a hook-pole. Each panel of needles is set or removed entirely as the maneuvers progress, so as to leave no trestles or needles exposed to chance drift.

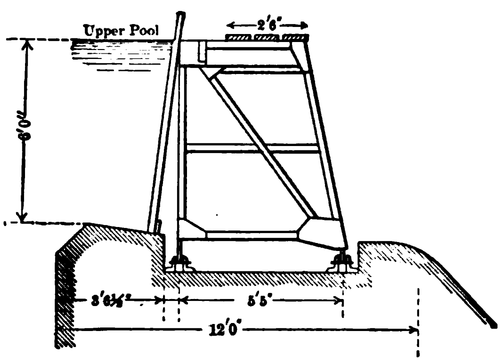


FIG. 201*a*.

With all movable crests, especially if of a trestle type, it is important to provide a masonry guard against injury from drift or ice when the parts are lying down, and where the current will be strong all pieces like floor sections, etc., should be arranged to be carried ashore, as far as practicable, leaving nothing loose on the crest. The design should also permit of maneuvers in case a sudden rise overflows the crest.

Arrangements for temporary cofferdams for repairs to the movable portions should be provided with these structures, the simplest form being flash-boards supported by removable pins.

A formula for the width of apron for a dam with a movable crest will be found on p. 375.

The cost of the movable crests just described was as follows:

- Drum crests of the Monongahela River, \$24.00 per foot run, or \$8.00 per foot of height (total height 3 feet), not including operating machinery.
- Boulé Crest of Dam No. 11, Muskingum River, \$13.30 per foot run or \$3.33 per foot of height (total height 4 feet).
- Shutters of Dam No. 5, Monongahela River, \$8.00 per foot run or \$2.66 per foot of height (total height 3 feet).
- Needle crests of the Kentucky River, \$10.00 per foot run or \$1.67 per foot of height (total height 6 feet).

* Up to Jan. 1, 1913, the movable crest of Dam No. 11 only had been used, owing to the necessity for keeping the pool above Dam No. 12 at a low stage during the construction of Dams No. 13 and 14. An experience of about six years at Dam No. 11 indicates that this form of crest can be operated safely, but the fluctuations in the pool level, necessitated by the maneuvers for passing small rises and by complications from drift, are a constant source of uncertainty to navigation.

Sluiceways and Drift-chutes.—With dams on rivers of small flood range some type of sluice is usually employed in lieu of a movable crest. The simplest form consists of a rectangular opening closed by needles, with intermediate piers if the sluice is long. For convenience in operation it is best to use hook-needles of square cross-section (see Fig. 209, p. 561), and to limit the head of water to about 6 feet. An iron or a wooden beam, removable in floods, is used to support the tops of the needles. This design will give a clear passage for drift, etc. The sluiceway can also be closed by a short section of movable dam, as described under the preceding heading, and also in Chapter V and elsewhere.

A common method of closure on rivers of this class consists of the ordinary vertical sluice-gate or shutter of wood or of metal. If small it is usually of the sliding type; if large, rollers must be used, and possibly counterweights.* The sluices should be located where ice and drift will not damage them, as the guides for the gates remain in place, and do not afford a clear waterway (unless the gates are of a considerable width) as does the needle sluice. The "Taintor" or sector gate (Fig. 202) is also used in place of the flat gate, as it does away with rollers and is more easily operated than the sliding gate. Its upstream face is curved as shown so as to transmit all pressure direct to the axis. The width is usually limited to about 20 feet. At Sterling, Illinois, is a large dam of this type composed partly of a fixed crest and partly of a series of Taintor gates operated from a foot-bridge.

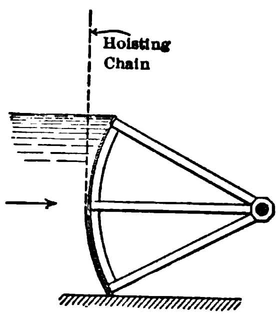


FIG. 202.—Taintor Gate.

On the River Thames in England and elsewhere examples are found of dams with part of the spillway as a fixed crest and part provided with a series of small wooden sluice gates, raised with a lever or with cranks and pinions, and operated from a walk. These are known as "draw-door weirs." As there is practically no drift on this river the stems and guides usually pass through floods without injury.

Special precautions must be taken to guard against undermining on the downstream side, as the rush of water from any type of sluice is of dangerous force, and if the structure is built on light material and improperly protected a washout will speedily occur.

Abutment and Protection.—The abutment is needed to give protection to the end of the dam, and to prevent the river from undermining it by seeping round at all stages and by scouring below or behind it in high water. It must therefore be carried into the solid bank so as to cut off the seepage—forming in fact an extension of the dam—and must be well protected against reactions below and against the danger that the river when running over it in flood may cut a channel in the surface of the bank and so "flank" the dam. Too much care can hardly

* The Stoney type of gate is described in Chapter VIII.

be taken in securing this end of the dam, as several cases have occurred where the river in a single flood has cut away the banks and found its way around the abutment, as will be described in Chapter X, under the clause "Flanking." This part of the work should be well advanced before the flow of the river is contracted to an appreciable extent by cofferdams or other construction connected with the dam. In two cases where this precaution was neglected the river cut into the unprotected abutment bank for a width of several hundred feet; in another case, the cutting necessitated a revision of the entire plan and the making of a new contract. The additional expense in all these cases caused a great increase in the final cost.

Wooden cribs filled with stone and closely planked on the river face have served as abutments in many of the early examples, and where sheet-piling and other suitable precautions were used to prevent the water leaking past they served well. The type, however, is much inferior to masonry, as the exposed timber decays and the face planking is often torn off or damaged by drift, etc.

Another type in use, especially with small dams, is a combination of a masonry superstructure above low-water line, resting on a timber crib. This crib is sheathed inside and provided with suitable sheet-piling along the upstream face, and is usually filled with loose stone. In one or two cases grouting has been used to fill the voids in the stone, with apparently satisfactory results. Special care must be taken, where the masonry is heavy, to see that the timbers have sufficient bearing power upon each other and at the base, and the outside of the crib usually has to be made solid to secure this. If built like an ordinary crib, with the stringers supported only by the ends of ties 8 or 10 feet apart, the latter will almost invariably give way and the whole structure will settle forward. It must be remembered that the overturning tendency of the earth backing produces a concentrated load along the face which may greatly increase the average vertical unit pressure, and this latter is rendered still more excessive by any forward settling of the structure.

Two general designs are used for masonry abutments, the U and the L (Figs. 203, 204, and 205). The latter is usually preferable, not only because its single wing wall has proved able to cut off seepage equally as well as the double wall of the U type, but also because its long tail wall provides a permanent protection for the toe of the slope just where it is most needed, and which in the other design must be provided by a crib or by riprap. Abutments which slope up the bank without a vertical face at the dam have also been tried, but have not given satisfaction where the currents were strong. There is too much risk of the paving washing out. This actually occurred in one case and the river cut around the abutment in consequence.

The top of the abutment is usually placed at the same level as the lock walls. The length of the wing AB need be only sufficient to protect the slope and to reach back into the undisturbed bank for a few feet beyond the line of excavation.

Experience has shown that when the excavation is properly back filled around it (a good firm-packing earth will serve for the purpose), and requisite precautions are taken when building to prevent leakage underneath, all seepage is practically cut off. Traces of leakage have occasionally appeared with some of the older constructions (see "Undermining," Chapter X), but experience has shown that unless they

FIG. 203.—U Abutment.

FIG. 204.—U Abutment.

increase, they are not necessarily dangerous. They should, however, be remedied if practicable. In case leaks of any magnitude occur, about the only temporary remedy is to throw gravel and earth above the abutment with the chance of its covering the inlet, and if permanent relief is not secured, it may be necessary later to excavate into the bank and try to find the cause. If the leak becomes dangerous,

FIG. 205.—L Abutment.

sand-bags or riprap may have to be thrown in to afford a base for the gravel and earth. Thus it is very important to use good sheet-piling or other secure means for a cut-off under the wing, and it should connect tightly with the piling or cut-off of the dam. It is also wise to use piles under all abutment masonry where on sand or similar light material, and in bad ground to drive sheet-piling part or all around the foundation, embedding the tops in a bottom course of concrete. If desired,

the wing sheet-piling may be continued past the end of the masonry and on into the bank, but rammed earth serves very well.

The upstream face of the wing wall is usually placed on a line with that of the dam, and the base is stepped up as it goes back. With concrete a vertical joint should always be placed just back of the main wall, as wings have a marked tendency to settle away, and temperature also is liable to produce cracks there.* If this is not done an unsightly break may occur. In some cases reinforcement has been built into the masonry at this point so as to prevent a break and the joint has been omitted. This method is of much value in tending to counteract any possible forward settlement of an abutment.

The base of the main wall is usually placed on a level with the base of the dam, as the chief danger to be guarded against if the foundation is porous, is undermining, but a foot or two more or less of elevation will rarely make much difference. The beginning of the slope at C may be placed where the drop of the water surface begins, or say on the crest line, and the elevation at D is usually a foot or two above the level of the lower pool. The wall may extend from 50 to 100 feet below the downstream face of the dam proper, depending on the lift, amount of reaction expected, type of dam, etc. With masonry dams with their sudden fall, and with movable dams with their strong currents during maneuvers, a longer wall is generally needed than with the more moderate-sloped crib dams. Plenty of heavy riprap should be placed all along the toe, as this is a very exposed point, and the undermining action is always strong there. The pieces may be from $\frac{3}{4}$ to 1 cubic yard and over, the heaviest being placed at the dam. Small riprap should be thrown in with it, as it helps to fill the voids and prevents the eddies from working in too far among the larger pieces. This toe protection should be continued for another 100 feet or so below the end of the wall, and as the currents below the wall are usually much less violent than at the dam, one-man riprap will usually prove sufficiently heavy. In the case of movable dams, however, it is better to use larger stone, as the volume of the current carries the rough water further than with fixed dams (see "Foundations," p. 547 and after). All this protection work should be carefully watched for a year or two after completion, and weak points should be promptly reinforced. Once the water has found a vulnerable spot, it may not take long for conditions to become dangerous, especially as the first damage is usually hidden from view until some break brings it to notice.

At some of the dams on the Kentucky River, where the ends abutted rock cliffs, a simple masonry facing, a few feet in thickness, and without any wing or protection wall, was used for the abutment. Examples of this type are occasionally met with elsewhere, and a similar construction has also been used for lock walls. (See Cuts on pp. 405 and 406 and Pl. 45*a*.)

* This peculiarity is said to have been in evidence even where the masonry was founded on rock.

Protection of Lock.—The foregoing remarks about the need of protecting the abutment from scour below and from cutting behind in high floods apply equally to the lock wall where exposed to the reaction of the dam. The danger is similar in each case, and similar precautions must be taken to guard against it. Examples of the disastrous effects of insufficient protection will be found in Chapter X, under the clause "Flanking."

Construction of Abutment.—The construction of an abutment is frequently a difficult problem, since it may have to be placed in the bank and the bank may be full of seepages. One method of overcoming this is to drive sheet- or round piling to form a rough cofferdam against the earth, and then to excavate as rapidly as possible inside the inclosure, and build up a foundation of concrete to the height that may be required. If the work is done quickly the concrete will be in place before much more slipping has occurred, and will permit the remainder of the masonry to be put in with more leisure. The piling should be driven a little distance from the building lines, as the earth will push it in more or less during the excavation. If the masonry is to rest on piles, they should be driven before the excavation is completed, so as to save any delays.

Another method is to excavate as deeply as possible without using sheet-piling, and then to take sharpened planks, and set them up in a close row against "rangers," or horizontal planks, which are kept in position by shores or horizontal struts, each side of the excavation being braced against the opposite side. The material is then dug out below the feet of the vertical planks, which are driven down by hand as far as possible, and the material from the center of the excavation can then be removed. Leaks may be checked by using straw, weeds, excelsior, or similar materials. This operation is repeated till the final depth is reached, when the masonry can be put in as above described.

Protection of Banks. (See also Part I, Chapter V.)—In a bank of very unstable nature it is best to make the finished grade to a flat slope, 2 or 3 horizontal to 1 vertical, and unless the seepage is very bad, it will be found that the bank will drain ultimately itself and become stable. It must of course be paved or riprapped, and if a U abutment is built (Figs. 203 and 204), the foot of the bank below should be protected with a toe of heavy riprap or a crib filled with riprap, or, better still, with a concrete wall. With rivers of high floods or strong currents this wall should extend 150 or 200 feet, and should always go to rock when practicable, or, if the foundation is of gravel or light material the base should be sunk as deep as possible and have a close row of long piles driven along its outer side, faced with heavy riprap. The bank above this protection and around the abutment should be paved with blocks with close joints, or with hand-placed riprap, and if the river is subject to high floods which will overtop the abutment the stones should be bedded roughly in Portland-cement mortar or in concrete where they will

be most exposed to the reaction from the dam. Any cutting of the bank at or close to the abutment during a flood may lead to slides and perhaps a flanking of the structure. (See Chapter X, "Flanking.")

Where the range of floods is very small the precautions need not be so elaborate, and in such cases a protection of piles and fascines, or of woven brush and riprap, may be sufficient, extending only a short distance below the abutment. This type is much used in Europe, and has been successfully employed on the Fox River in Wisconsin.

The bank above the abutment should also be graded and riprapped, but as it is not exposed to unusual currents, as is the bank below, but only to wave-wash and moderate velocities, a layer of paving or one-man riprap laid on spalls or gravel and extending for say 50 feet above the masonry, will be a sufficient protection for all ordinary cases. Its toe should be protected by riprap to prevent undercutting, and additional riprap should be thrown in at the corner of the abutment, as strong eddies frequently occur there.

In some cases it has been necessary to continue the protection of the banks, both below the lock and below the abutment, until it has reached a length of several hundred feet. As a rule, however, it will be sufficient for the first season to rely on the riprap or paving below the abutment as just described; but if the banks are "rotten," that is, if they are sandy and liable to slips, a good supply of stone should be kept on hand for emergencies. In the following season the bank, where cut away by the water, should be graded and riprapped, and this process should be ultimately continued as far downstream as the wash and eddies from the dam show it to be needful. This will range from 200 to 1200 feet or more, depending on the lift of the dam and other circumstances. A rough practical rule may be applied that for each foot of lift of a fixed dam 100 feet of bank, if alluvial, will be ultimately affected by the washing, and for each foot of lift of a movable dam, 150 feet. Gravel or clay banks will be affected to much less extent. (See also p. 338, "Acquisition of Land.")

Unless the bank is very hard, the riprap or paving should never be placed on the bare soil, but always on a bed of spalls or large gravel, 6 inches or more in depth. Where this latter covering is omitted the ripples and waves from the dam, which form the agency by which the banks are cut, play through the openings in the stones and wash upon the soil with a force very little diminished, and soon carry away the particles of earth. On Green River, Kentucky, and Little Kanawha River, in West Virginia, the two methods were tried side by side upon a bank just below a dam, and the portion protected by riprap alone was eaten away almost as though no stone lay upon it, while the portion on which spalls had been placed was unaffected. Large stones or paving are undermined as easily as small ones, and the use of spalls or gravel appears to be the only certain and economical method of avoiding settlement.

The protection of the banks is a matter which is usually neglected until some action is absolutely necessary. This is unfortunate, as by that time the bank has usually become badly washed and must always remain an eyesore. By applying the remedy at the proper time, the ultimate expense is usually lessened and the disfigurement of property is avoided.

Several notable examples of the results of lack of bank protection are to be seen in the United States. During the control of the Little Kanawha River, West Virginia, by a navigation company, the rough water below some of the dams ate into the banks behind the adjoining locks until it had excavated areas of several acres below and behind the land walls. In one case this resulted in the river cutting its way around the upper wing wall into the basin during a flood and causing very serious damage.

Leakage of Timber Dams.—A timber dam usually becomes very leaky as its age increases, because the water gradually enlarges all crevices and holes through which it can pass. On many rivers this loss of water is serious, as the pool may fall 3 or 4 feet below the crest in a dry season, and navigation becomes embarrassed. Temporary relief may be obtained by throwing in a generous amount of boiler ashes, fine gravel, heavy sawdust, or similar material along the upstream face of the dam; the water will carry the particles into the crevices and gradually close them. The first rise, however, usually reopens them, and it is often not until late in the autumn, when the fallen leaves are carried down the river and into the leaks, that the loss of water ceases.

It has usually been found that the leaks which were the most copious and the hardest to close occurred through the upstream vertical sheathing planks at and near their tops, which in some cases have been found badly worn for a length of 3 feet or more, or to a depth of 6 or 7 feet below the pool. The upstream slope sheathing is also apt to wear and leak badly, but is easier to repair. For these reasons, it has been suggested to use in both these places a layer of 4 to 6 inches of concrete instead of the plank, as it would be watertight and not wear away. It could be laid upon a facing of plank studded with nails to secure adhesion, and should be provided with reinforcing rods as a protection against damage from drift, etc.

In repairing crib dams the upper portions have in several cases been removed and replaced with a concrete covering several feet in thickness. On the Kentucky River this thickness was made 4 feet.

General Remarks. (See also "General Remarks," p. 554 and after.)—The chief advantages of fixed dams as compared with movable ones lie in their cheaper first cost, the less expense for care and maintenance, and the opportunity of securing water power. The first-named advantage is of less importance than formerly, except where a temporary structure is to be built. The constantly-increasing price

of timber has made the old type of stone-filled crib dam almost as costly in many localities as would be a dam of concrete, and this in turn may prove almost as costly, if flood damages are to be reckoned with, as a movable dam, or a fixed dam with a movable crest. The second advantage, however, is a very important one, as a fixed dam, especially if of masonry, should need no attention beyond very occasional repairs, while a movable dam requires a special force of men during the season of navigation and is much more liable to injury. The third advantage is one of growing importance, but is shared in part by some of the types of movable dam, especially where the width of river is not great. On the Bohemian Elbe a system of movable dams was planned about 1907 in which the creation of water power was considered as important as that of navigation.

The chief comparative disadvantages are that the fixed dam is an obstruction to boats during floods when they could have passed up or down without hindrance had a movable dam been used, and that to a certain extent, they permanently raise flood levels, especially in moderate stages. The first-named disadvantage is becoming of minor importance, although freedom of navigation was originally one of the chief influences which led to the introduction of movable dams. The number of floods per annum which will afford draft enough for open-river navigation on streams canalized for the considerable depths of modern commerce—as 10½ feet on the lower Seine and 12 feet on the river lines of the New York State Barge Canal—is small, and the tendency on minor rivers is also toward a constant increase of depth. Under these conditions the movable dam has become little more than a regulator of pool levels, especially as the currents during the so-called navigable floods are often swift and sometimes dangerous to boats.

The second disadvantage—the change of flood levels—has become the more important one, especially for rivers of low banks or which flow through valuable property (see also p. 521). While for streams of high floods the change of level may be very small, it has often led to claims for and payment of heavy damages. The fact that an obstruction had been placed in the river has been held by the courts to be clear proof that the flood level had been raised thereby, and for this reason the adoption of movable dams on certain rivers, especially in Europe, has become imperative for self-protection. Their additional cost, however, is often modified by the possibility of using higher pool levels than would have been advisable for fixed dams, as the water can be controlled so as not to inundate property at moderate stages. (See "Saturation of Land," p. 389.) Thus on the Upper Seine the pool levels just above the movable dams are in many cases within 2 or 3 feet of the tops of the banks, but the levels can be kept there until the dams are opened and the river free. If fixed dams had been used and placed at similar elevations a rise of a few feet would have caused serious flooding.

The effect of fixed dams on the passage of silt has been treated on p. 389.

Weirs on the Nile.—Between 1898 and 1901 two weirs or fixed dams were built in the Delta of the Nile, below the well-known "Delta Barrage" which had been commenced about 1844 by the French. Their object was partly to reduce the head (14 feet 3 inches maximum) on the Barrage which had been a cause of constant anxiety owing to faulty construction, and partly to supply certain irrigation canals. The dam across the Rosetta mouth is 1640 feet in length, and that across the Damietta mouth, 1370 feet. The lift is 10 feet 6 inches, which is overcome by locks of the usual type, with chambers 56 feet in width. The foundation is of river sand or silt of very light quality, and all the masonry, including that of the locks, was constructed by grouting, a process which was adopted partly to save time and partly because of the supply of unskilled labor. The results were found to be highly satisfactory when some of the masonry was cut out for examination.

The main feature of the dams, as shown in Figs. 199 and 200, p. 519, consists in the core walls, which are about 10 feet thick and 28 feet high. They were built, except the upper few feet, of rubble stone filled with cement grout. A trench was first dredged across the river to the full depth required; wooden boxes, supported by barges, and formed of 3-inch plank properly braced, were then moored in place; the rubble was thrown in, and cement grout next introduced through pipes till all voids were filled. The box sides were set up in pieces like sheet-piles 5 feet wide, each piece being driven a short distance into the sand to prevent the escape of grout. When all the pieces were braced and secured in place, a canvas or sacking lining was fastened by divers over the inside and matched boards 1 inch thick were nailed on to protect the lining from injury when the rubble was thrown in. The grouting pipes were placed along the axis of the box in one row about 8 feet centers, and consisted of 5-inch perforated pipes with a 3-inch pipe, not perforated, inside, reaching to near the bottom, and made in short lengths. This pipe was to ensure the delivery of the grout in a thick stream, and to prevent its dispersion by the water when poured in. When all was ready, the rubble was thrown in and the grouting commenced, the height of the liquid being registered by floats in the outer pipes. It was found necessary to let the cement set when a height of 6 feet had been reached, as otherwise its weight forced leaks under the boxes. As the grouting proceeded, the 3-inch pipe was raised so as to keep the delivery at the proper height. The blocks of masonry were allowed to stand for 24 hours after completion, when the box was taken apart and set up for the adjoining block. To avoid all possibility of leakage, a vertical groove was left in the joint of each block, and was afterwards filled with grout.

The blocks were made in lengths of about 32 feet, in water from $19\frac{1}{2}$ to $24\frac{1}{2}$ feet deep, and at the average rate of 65 yards a day, a block containing from 208 to 221 cubic yards. About $4\frac{3}{4}$ barrels of cement, or 40 per cent, were used per cubic yard; with no leaks, $3\frac{1}{2}$ barrels, or $37\frac{1}{2}$ per cent of the mass, would have sufficed.

The total cost of the two works was $2,100,000, or about $700 per lineal foot. This includes the cost of the locks, etc. The cost of the grouted masonry was about $8.90 per cubic yard.*

Atchafalaya Dams.—Two submerged dams of brush on the Atchafalaya River in Louisiana, have been described on p. 239. Their design is of interest here on account of their treacherous foundations and the large amount of water passing over them in floods. The cross-section of one of them is shown by Fig. 120*a*, p. 239.

Ice and Fixed Dams. (See also "Ice Pressures," p. 372.)—When the ice of a frozen river breaks up and is carried out by floods it usually becomes stranded at one or more points in its passage and forms a blockade or "gorge," which remains until the pressure of the rising water behind breaks it and carries it in masses downstream. These do not often cause injury to works of navigation, one reason being that at such times there is usually an ample depth of water and the ice is carried past, doing little harm. A dangerous effect is liable to result to a fixed dam, however, when the flood is small but sufficient to carry out the ice, and if the pieces are thick they may strand on the top of the dam if the crest is sloping, and form a gorge there. Such a gorge has occasionally caused serious injury before the water carried it out. Thus it has happened on some of the crib dams of the Monongahela River in Pennsylvania (built with sloping crests) that the ice has torn off a portion of the timber work, and the water flowing through the opening has made a gap which threatened the destruction of the dam. In one case a gap 120 feet in width was formed in the structure, and the current scoured to 30 feet below the foundation timbers. The depth of water on the crest when the damage began was several feet. Similar experiences were had on the Schuylkill and Susquehanna Rivers. For this reason in some dams recently constructed the crest has been made flat and several feet (usually 6 to 10) in width, instead of using a rounded crest or a crest which sloped up to an apex (see Pls. 54 and 57). It has been found with this type that when the cakes of ice meet the crest they do not become stranded upon it and then pile up one upon another, but they either lie there horizontally until the rising water carries them over, or they turn on edge, sink down and then lie flat against the upstream face of the dam until a considerable mass has accumulated, the water flowing over its top and the passing cakes of ice breaking off any projections. Under such conditions it has been found that gorges never form upon the dam.†

Formulas for Discharge Over Dams.—For convenience of reference the formulas in customary use for computing the discharge over dams are given here. The results obtained will be only approximate, since the formulas are based on experi-

* Minutes of Proceedings Inst. C. E., 1903–1904. Papers by Messrs. G. H. Stephens and F. A. Hurley.

† See also various papers of the International Congress of Navigation, 1908.

ments with comparatively small units and the coefficients vary somewhat with the shape of the crest and the depth of the overflow.

Free Discharge.—Where the crest of the dam is above the level of the water on its downstream side and neglecting the small effect of the end contractions if such are present, the discharge Q in cubic feet per second is (Francis formula):

$$Q=CLH^{\frac{3}{2}}, \qquad (1)$$

where C is a coefficient, L is the length of the dam in feet, and H is the head measured from the crest to the surface of the water just upstream of where the "break" over the dam begins. Mr. Francis' experiments were made with a thin-edged weir and the value of C was found to average 3.33. Later experiments have shown that for weirs of flat crests the value of C is reduced, a broad and level crest giving a value of 2.6 or slightly less, while for crests with easy approach and overflow C may increase to 3.5 or more.*

If velocity of approach is to be considered, as should be done when computing flood discharges, the following correction should be applied:

$$H=D+\frac{v^2}{2g}, \qquad (2)$$

where D is the head on the crest when there is no velocity of approach (equivalent to H in formula (1)); v = the velocity of approach in feet per second, and $g=32.2$.

Submerged Discharge.—Where the crest of the dam is below the level of the downstream pool, thus having in a measure an obstructed discharge, the Herschel formula may be used:

$$Q=3.33\times L(aH^{\frac{3}{2}}), \qquad (3)$$

applying where necessary the same correction for velocity as given above. The notation of equation (3) is similar to the corresponding notation of (1), and a is a coefficient the values of which may be obtained from p. 140 of the paper of Mr. Horton referred to in the footnote, or from other works on Hydraulics.

Fixed Dams and Flood Levels.—The general effect of a fixed dam is to raise the original flood levels in its immediate vicinity and to some distance upstream. A dam of high lift and of a short length as compared with the natural width of the river has of course a greater effect than one of opposite features. The effect, however, is not as great as might be at first supposed, since as long as there is

* For a general summary of experiments and results see Weir Experiments, Coefficients and Formulas, Robert E. Horton. Paper No. 200, U. S. Geological Survey, 1907.

any fall over the crest the velocity upon it is considerable, and a given flow of water will pass over it with a considerably less relative depth than would be needed in an unobstructed channel. The height to which a flood will rise is usually investigated by first ascertaining the depth on the crest which a given discharge will require. For this the Francis or Herschel formulas may be used (see p. 534) according as the discharge is free or over a submerged crest respectively. Starting with this height as a basis the surface of the water at various points upstream is ascertained by using the customary formula for flow in open channels given on p. 71.*

Drowning of Fixed Dams.—The term "drowning" or "drowning-out" is used to indicate the fact that at a certain stage of flood the fall over a fixed dam usually disappears, and the elevation of the water immediately above and below it coincide with the general slope of the river. Where the dam is of high lift or is comparatively short such a condition may not obtain, as at certain of the 18-ft. lift dams of the upper Kentucky River, where with a depth of 12 feet on the crest a fall of 6 or 7 feet has been still in evidence. Similarly at Dam No. 2 on the White River (Arkansas) one flood caused a depth of 18 feet on the crest with an accompanying fall of nearly 4 feet. On the other hand, where the dam is wide and of moderate lift (especially if there is a narrower portion of the river just below) it will become drowned at comparatively moderate stages, as at Dam No. 2, Green River, Kentucky. This dam has a lift of about 15½ feet, and with a depth of 6 or 7 feet on the crest the fall usually becomes obliterated. In this particular case the action is assisted by the fact that the next dam below is about 60 miles away, so that the water has to gather below the dam in order to obtain a sufficient hydraulic head to carry it through the pool below. It has been observed that for average cases the rise on the crest and the rise immediately below continue about equal in amount for the first few feet and that after that (depending largely on the length of the lower pool and the neighboring configuration of the bed) the water on the downstream side will rise from 2 to 3 feet for each rise of 1 foot on the crest. If, however, the lower pool is very short this acceleration of rise does not take place.

* See also "Rivières Canalisées," De Mas, p. 8, and Mr. Horton's paper referred to in the footnote on p. 534, for shorter methods of finding approximate backwater elevations.

COST OF FIXED DAMS

Location.	Lift.	Length.	Width of Base.	Average Height.	Approx. Contents Cu. Yds.	Cost per Cu. Yd.	Total Cost.	Cost per Foot Run.	Remarks.
Stone-filled Timber-crib Dams.									
Kanawha River, W. Va., Dam No. 2, 1887	12' 0"	524'	38'	20'	14,750	$6.35	$94,000	$180.00	Cost of dam only. Is on rock foundation.
Do	.....	....	...	...	.....	7.58	111,500	212.00	Cost includes abutment, bank protection, etc.
Fox River, Wis., Princeton Dam, 1897	about 4'	180'	30'	10'	1,420	7.80	11,083	61.50	Cost includes abutment, bank protection, etc. Dam is of cribwork and stone filling. Cost of dam only, $3.70 per cu.yd. Built on sand.
Fox River, Wis., Berlin Dam, 1893	3' 4"	200'	30'	10'	1,580	9.35	14,747	73.70	Cost includes abutment, bank protection, etc. Same type as at Princeton.
Kentucky River, Ky., Dam No. 7, 1897	15' 0"	351'	60'	20'	13,000	2.00	25,780	73.65	Cost of dam only. Is on rock foundation.
Green River, Ky., Dam No. 5, 1899	14' 0"	281'	46'	16'	7,000	3.35	22,780	81.90	Cost of dam only. Built on sandy gravel.
Riprap Dams.									
Rosetta and Damietta Weirs (combined). (Figs. 199 and 200, p. 519.)	10' 10"	3,015'	See Figs. 199 and 200		.....	.....	2,100,000	700.00	On fine sand and silt.
Chenab. (Fig. 201, p. 519.)	13' 3"	4,000'	See Fig. 201		.....	.....	1,300,000	325.00	On fine sand and silt.
Sone	10' 0"	12,500'	.....	...	.....	.....	870,000	69.50	On sand with some gravel.
Rupar	13'-14'	2,400'	...	...	.....	.....	555,000	231.00	On sand with some gravel.

Masonry Dams.—The cost of masonry dams, including cofferdams but no permanent protective works, usually varies from $8 to $12 per cubic yard.

CHAPTER V.

MOVABLE DAMS.

History.—A movable dam differs from a fixed dam in that the dam proper is so designed that it can be lowered or raised as may be needed, its principal object being to provide slack-water in times of low water, without forming an obstruction to navigation in moderate stages, or to floods. The last is an advantage of great importance with rivers of low banks, as a movable dam does not increase the dangers of inundation.

With the exception of the bear-trap dam, which is an American type, the invention of movable dams is due to the genius of the French engineers. Prior to 1830 the fixed or stationary dam was the only one used for navigation purposes. These had been in use on the Lot since the thirteenth century, and, with the introduction of locks in the fifteenth century, had been constructed on many rivers, but they were open to the same objection that exists to-day—the principle was in part unfavorable to navigation. Dams with small navigable passes had been used, but the ascent of the pass was always very laborious. These passes or openings were closed either by beams lying one upon another, supported against piles or piers at the ends, or by planks resting against a sill in the river-bed at the bottom, and against a beam spanning the opening at the top. When it was desired to open the passage the beams or planks were removed, either one by one, or simultaneously, and the water rushed through with great violence. Sometimes they were used for the purpose of producing artificial floods, by damming up the whole river until the level of the pool above the dam had been raised to the desired height, when, by their sudden removal, the water escaped and carried rafts or boats over the shallow places below. The operation of letting out the water was called "flushing" or "flashing"; in this country on log streams it is called "splashing."

On the Yonne in France this system of navigation was in use as late as 1880, and on parts of the Upper Seine until 1898, the flashes being commenced at prearranged times at the upper end of the river and continued downstream successively as the boats neared the dams. The passes were 25 to 40 feet wide.

NOTE.—Part of the matter in the following chapters, with some of the illustrations, is taken from a paper on "Movable Dams," by B. F. Thomas, published in the Transactions of the American Society of Civil Engineers, June, 1898, and republished by permission of that Society, and others of the illustrations are republished by permission from "The Design and Construction of Dams," by Edward Wegmann.

Tables of lifts, etc., of various dams will be found at the end of the book, and Pl. 46 shows the location of the principal rivers in the United States possessing movable dams.

Falling gates or shutters, supported by props when upright, were built across a fixed dam on the river Orb, in France, in the eighteenth century, forming the first attempt at placing movable weirs on fixed dams.

The first distinct type of movable dam was erected in 1818 on the Lehigh River, in the United States, and was called a bear-trap dam. It consisted of two wooden gates revolving on horizontal axes at the floor level. The downstream gate pointed upstream, and the upstream one pointed downstream, the latter resting on the edge of the downstream gate when raised. The dam was operated by water running under the gates through culverts and forcing them up. The example was not copied, and until recent years the type remained practically unknown. It will be found described at length in Chapter IX.

In the year 1834 M. Poirée, an eminent French engineer, invented the needle dam. This ushered in a new era in navigation, and this type of dam soon multiplied and was improved and modified, and other inventors came forward with new ideas, some good, some bad, until to-day there are numerous systems from which to choose.

The invention of movable dams was only arrived at after long discussion of ways and means for more successfully operating the apparatus used for closing the chutes in the old stationary dams. The use of needles was then already old. The problem to be solved was how best to widen the passages to accommodate the increased requirements. The experiment of supporting the tops of the needles by a rope was tried, and was in a measure satisfactory for lifts of 2 to 3 feet on passes of considerable width. The rope, which was braced to the downstream side of the foundation by strips of wood, was tied to a pier by one end while the other was wound on a windlass. To open the dam it was only necessary to release the line at the pier, when the whole set of needles would float out, being attached to the rope beforehand, as were also the braces.

This was the status of improvements in fixed dams when the first movable dam was constructed, and it was only natural that iron trestles should supersede ropes in needle dams, that gates sliding on these trestles should later on replace the sluice-gates of the old chutes operated from an overhead bridge, that the swinging wickets formerly used to increase the heights of stationary dams should form the dam itself in after years, and that *poutrelles* or horizontal timbers hinged together and supported on trestles should form the curtain dam that was to come into use.

Classes and Kinds.—Movable dams may be divided into two general classes: (1) the non-automatic or those requiring extraneous power for their maneuvers, and (2) the automatic or those operated by the force of the water. Among the first class are the various types of trestle and wicket dams, like the Poirée, Chanoine, Boulé, Cameré, etc., while the second class comprises the several forms of bear-traps, drum-wickets, etc. The first class is the principal one applied to navigable rivers (although several examples exist where a weir or drift chute of the second class is

used in conjunction with a dam of the first class), and its application until recent years has been confined largely to the wickets of Chanoine and the trestles and needles of Poirée.

The forms of closing are many, and frequently vary on the same dam; for instance, the pass may be of wickets and the weir of needles, or the reverse may be the case; while more than one type has been applied, even on the same part of a dam; for instance at the Suresnes dam, in France, the pass is closed by trestles supporting alternate bays of Boulé gates and Cameré curtains.

In the needle dam the water is dammed up by vertical planks, called needles, resting against bars connecting the trestles at the top, and against a sill in the river-bed at the bottom. The trestles are spaced from 3 to 10 feet apart, and when not in use lie down across the stream, being protected from injury by the sill. A walkway connects them when standing.

The Chanoine wicket is an upright shutter, hinged near its middle to a horse connected to the floor, and also to a prop which rests against a shoe on the floor. The displacement of the end of the prop permits the wicket to fall with the current.

The Boulé gate and Cameré curtain replace the needles by small gates and by curtains respectively, resting directly against the trestles or against uprights leaning on the trestles.

In the overhead bridge dam, where the closure is always composed of such gates or curtains, the supports are drawn up to the bridge when not in use, while their feet rest against a sill in the river-bed when in use.

These are the principal types of dams, and practically the only ones employed. Many modifications of each one have been suggested, but rarely, if ever, put into practice, and therefore will not be described.*

Operation.—As the river rises a sufficient amount of the dam is moved or opened to allow the additional flow to pass without raising the water level just above the dam; as the river falls the dam is closed in proportion and the water level is maintained. The water below begins of course to rise above its normal level as soon as any part of the dam is opened, owing to the increased discharge. If there comes a flood sufficient to allow boats to navigate the river without needing the additional depth provided by the dam, part or all of the movable portions are put out of the way and the river is then "open." The dam is replaced in proportion as the flood recedes, and navigation is thus always maintained.

The exact level at which the water is held when the dam is in operation varies somewhat with the conditions. At the Suresnes dam, which holds the pool through Paris, the operators have instructions to maintain the upper pool level within about 1 inch of variation. This requires close attention and much regulation, and

* Most of these modifications will be found described in a paper on Movable Dams by B. F. Thomas, Transactions, Am. Soc. C.E., June, 1897, and later and more comprehensive information is given in "Die beweglichen Wehre," by Professor K. E. Hilgard (Wm. Engelmann, Leipzig, 1912).

while an exact level may be maintained at the location of the dam, the height necessarily increases upstream during any flood owing to the slope created by the latter. In America the regulation is usually much less close, and the movable dams are generally designed to permit an overflow varying from a few inches to 1½ feet. This affords a spillway and saves much labor.

Where the dam forms a harbor for a city, it is usually maneuvered gradually on the approach of a flood so as to avoid as far as possible any sudden changes of water level.

Height of Lift.—The lift of movable dams has, until recent years, been very moderate. It was the belief of the earlier engineers, who did not possess the appliances of the present day, that all parts must be small enough to be maneuvered by hand, and the head of water in consequence had to be small. Thus on the first needle dams the head was limited to about 4 feet. As experience was gained, however, the lifts were gradually increased, until we find examples to-day of needle dams with lifts of more than 12 feet, and a curtain dam at Poses, on the lower Seine, with a lift of nearly 14 feet. Several Boulé dams are also in existence with lifts of 10 to 12 feet, and the bridge dams of the Mohawk River line of the New York State Barge Canal have lifts from 8 to 15 feet, with depth on the sills from 16 to 20 feet. (See p. 632 and after.)

The lift of a Chanoine dam, however, has rarely exceeded 8 feet. No recent examples of this type are to be found in Europe, where other methods of closure have come into use, and in consequence the Chanoine wicket has never received there its possible development. There appears to be no reason, however, why it could not be used for greater lifts than heretofore, since this would mean simply the application of heavier parts to hold back the water, and heavier machinery with which to perform the operations.

One of the chief objections to movable dams has been that their cost has been very great for the amount of river made navigable thereby; in other words, the lift attained has been small in comparison with the expense, and their success cannot be considered complete until they have been applied to lifts at least equal to those which would have been given to fixed dams at the same points, and at not much greater cost.

General Design.—The designing of a movable dam is one of the difficult problems of engineering, and requires in order to secure the best results not only a thorough knowledge of construction, but also a practical acquaintance with the operation and effects of movable dams in all their features.

Every such dam should fulfill as far as possible the following conditions:

(1) The head of water sustained should be not less than that advisable for a fixed dam at the same point, so as to reduce to a minimum the number of dams required.

(2) Injury to property should be avoided, and flood conditions should not be changed unless for the better.

(3) The dam should be capable of being operated by few employees and appliances, both in lowering and raising, under full head and in whole or in part, and without much risk to the operators.

(4) The crest should be submersible to an extent sufficient to take care of the flow at ordinary stages without requiring much maneuvering.

(5) The leakage should be reducible to a very small amount.

(6) The parts should be complete in themselves as far as possible without the introduction of additional means for keeping the pool full, even in low-water seasons.

(7) The first cost should not greatly exceed that of a fixed dam for the same location, and the cost of operation and maintenance should be moderate.

In general, movable dams are constructed in two or more sections, one for navigation, called the pass, and one or more for the passage of surplus water, called the weir. The object of the latter is to provide a means for passing small rises without having to handle the heavy appliances of the pass. On wide rivers with two or more weirs to the dam they are usually made of different heights, affording more facility in maneuvering. When the flood has reached the full discharge capacity of the weir or weirs, and is still rising, the pass must be lowered, and if the dam has been properly designed there will then be enough depth of water on the pass sill to allow full-draft boats to cross it in either direction; that is, the contracted current will not be swift enough to embarrass up-going craft.

The dam is usually placed opposite the lower end of the lock, but where the walls are long it is sometimes placed near the middle. In some examples where the lock was on a porous foundation, it has been placed at the upper end, so as to avoid creating under-pressure beneath the chamber. In such cases it is usually desirable to provide a guard wall on the river side above the lock in order to shelter boats from the current drawing towards the dam. With the dam at the lower end of the lock, experience has shown that no guard walls are needed on the river side either above or below. In America it is usual to provide upper and lower guide walls on the land side, as they facilitate greatly the movements of boats, but in Europe guide walls of any kind are very rarely found. The pass and the lock are almost always located on the channel side and adjacent to each other, thus placing the currents created by the weir, which may be required to be opened wholly or in part while the pass is still in position, as far from the entrances as possible.

The choice of type is usually decided by the characteristics of the river, and the results of experience in America and in Europe seem to show that for rivers of slow rises needles, gates, or curtains are well adapted, the first named being also used on small rivers of quick rises and where the low-water discharge is small. In this case the weir is usually closed with Chanoine wickets, although needles are

also used. For large rivers subject to high and sudden floods the Chanoine wicket has been usually adopted in America, and on the whole seems best adapted to such streams. Bear-traps and drum-shutters have not been used except as weirs to regulate the pools. The special features of the various types will be described in the chapters following.

Extended investigations made in Europe during 1905 in connection with the canalization of various rivers for the Barge Canal of the State of New York showed that the experience of more than fifty years with movable dams had indicated the superiority of two chief types, the needle and the gate dam. While the Chanoine wicket had proved its value, it had been found that its rapid action was not necessary for the comparatively mild regimen of European rivers, and that needles and gates could be maneuvered with sufficient quickness to regulate their floods. These two types, moreover, were the most economical in size of foundation and in simplicity of movable parts, and were free from the objections of difficulty of handling and the sudden release of a large volume of water, inherent in the Chanoine wicket type. Needles and gates had also proved their ability to meet conditions found on rivers of high floods, as on the Moldau, and the general opinion of engineers appeared to be that these two types would henceforth be the standards.

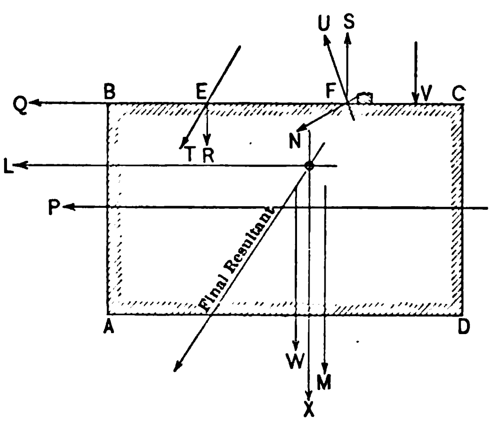


FIG. 206.

Calculations for Foundations.—The calculations for the foundation of a movable dam are based on the same general principles as given for lock-walls (see pp. 360 to 368) combined with the strains from the movable parts. Thus let $ABCD$ (Fig. 206) represent a section of a foundation for a needle dam. The forces acting per foot run are:

P = pressure of water on CD, CD in this instance being supposed vertical.

V = vertical water-pressure on the apron above the sill.

N = pressure from the needles against the sill.

T = thrust from downstream leg of trestle.

U = upward pull from upstream leg of trestle.

W = weight of masonry.

It should be noted that the water-pressure P acts a little below the center of CD, as a graphic representation of its elements would be a quadrilateral.

As described on p. 361 and after, W must be adjusted for underpressure if necessary, according to the material of the natural foundation. The pressure V will be a function of the difference between pools, if the lower pool is above the level

of BF. In this case the lower pool will also cause a downward pressure on BF, which may be combined with V. Resolve T, N and U into vertical and horizontal components, assuming N to act upon the same axis as T and U. We thus obtain a horizontal force Q, acting along FB, a downward force R, acting at E, and an upward force S (the algebraic sum of the vertical components of N and U), acting at F. Combining Q and P we obtain as their resultant the force L, and combining R, V, and W, we obtain their resultant M. Lastly, combining S and M we obtain the force X.

We thus have to deal finally with the two forces L and X, whose resultant should lie within a certain zone of the base as indicated on p. 368 and after.

Similar methods can be applied for calculating the foundations for other types of movable dams.

Area of Opening and Elevation of Sills.—In improving rivers which carry sediment (remembering the law that the cross-section of flow always tends to be proportionate to the discharge and its accompanying effects) it is usually desirable to provide about the same area for the dam as existed under the unimproved conditions. If the area is reduced, there will be a corresponding tendency to a contracted flow and swift currents which may embarrass navigation, while if it is increased the river will tend to fill up with sediment the excess area, indicating that it was unnecessary for the flood discharge. To fulfill theoretical conditions, the new areas should be proportionate to the natural ones at all the different stages of water; this is partially obtained by modifying the areas by the maneuvers of the dam. Above the stage, however, at which the dam has to be lowered entirely there is usually more or less unavoidable discrepancy of areas, due in some cases to the length deemed advisable for the weir, in others to that for the pass, etc. The sum total at the higher stages, however, should approximate the original area for the reasons given above, making allowance where desirable for the effect on the discharge of the new hydraulic radius as compared with the old. It is usual to include the lock in the area of discharge, as it is customary to open it if high or prolonged floods are expected (see p. 486) in order to avoid too great a deposit of sediment in the chamber. In spite of this, high and prolonged floods on some American rivers, as on the Kanawha, have been known to silt up the lock chambers half way to the top, and they have then to be scoured out by raising part of the dams. If the lock gates were kept closed, dredging or other means would have to be resorted to. Where the amount of sediment is small, as on some of the French rivers, this practice of opening the lock chamber has been abandoned, as it was considered undesirable to maneuver the gates against any head, and it had been found that the deposits could be cleared away sufficiently by the use of the valves.

The elevation of the pass sill is decided by the mean position of the river-bed and the navigable depth desired. Its top may be placed at the average elevation

of the natural bed, so as to secure the maximum area for a given length, or else about on the line which will connect the crests of the nearest bars above and below. It must not be higher than this line or it will become an obstruction at the stage occurring just before the raising of the dam, and when the lock is out of service, a condition not uncommon during the winter and spring, when it is not always advisable to raise the dam on account of ice or the expected approach of floods.

The sill of the weir is usually placed higher than that of the pass, and its elevation depends on its relation to the length of spillway, as shown in the next clause. Where several weirs are employed on the same dam, their sills should conform approximately to the natural bed of the river. If placed too low they will be covered with sediment at each flood, and this may cause trouble when the dam has to be raised. The portions on the convex bank naturally tend to become silted over in any case, and instances have occurred where a weir sill has been covered with 5 to 7 feet of deposit during a flood lasting only two or three weeks. Such a deposit, if of sand, can be disposed of easily by raising part of the dam and washing it away, but gravel is more troublesome, and for that reason weir sills in a gravel-bearing stream should be placed at or somewhat above the natural bed. Even with this arrangement the recess behind the sill will catch more or less of the gravel, and this sometimes becomes packed so closely among the movable portions of the dam after it has been lowered that it is difficult to raise the parts. The depth of the recess should therefore be kept as small as practicable.

It is of importance that the construction should cause no lowering of the original low-water level, as if this occurs it may start scouring on the shoals above, as described on p. 67 and after. Usually there will be little danger of this unless dredging of the shoals is employed, as in floods there would be no tendency to scour greater than existed before, and in low water, except during winter time or repairs, the dam would keep the pool full. The pool levels, moreover, are usually determined with a view to avoiding dredging, so that if no shoals are removed the low-water level will be maintained by them as before. Where they are dredged, however, so as to lower their crests, the question of the effect on the low-water level must be carefully investigated. (See also p. 637, at top.)

It may be of interest to note here gradual change of the functions of the pass brought about by the increase of navigable depths. In the early history of movable dams, when 4 feet were enough for navigation and much traffic was carried on in rafts, the pass was an indispensable adjunct for commerce, and was lowered whenever there was a flood high enough to let boats and rafts go through it and avoid the delays of locking. As the required depth increased, there were fewer days on which open navigation could be secured, and the movable portions of the pass became heavier and less easily handled. The needs of navigation grew steadily, and

in 1885 a depth was created in the lower Seine of $10\frac{1}{2}$ feet, and, beginning about 1912, of 12 feet on the Mohawk River section of the New York Barge Canal. With these depths the use of the pass for navigation practically disappears, not only because floods of the necessary volume are rare, but also because at such stages the swiftness of the current makes it difficult, and often dangerous, to handle the large boats in use. (See also " Dams on the Lower Seine," p. 558.) Under such requirements of depth, therefore, the pass becomes an enlarged weir, and serves only to facilitate the passages of floods.

With the Mohawk River system of movable dams above referred to and described at greater length on p. 632 and after, the sill at each dam was placed on one level all across the river, and at about the average elevation of the river-bed at the location in question. The net area of opening was made such that when the river rose to the top of the lock walls the cross-section available for discharge was about equal to that which had existed prior to the improvement at the same stage. Above this level a similar proportion obtained, while below it the proportion of artificial to natural area gradually increased, owing to the rectangular limits of the one as compared with the curved limits of the other. The area of the locks was not taken into account, as owing to the swift currents in floods and the severe ice conditions it would not be safe to leave the lock gates open during the winter.

Comparative Lengths of Pass and Weir.—In determining the lengths required, the chief conditions to be satisfied are that the pass shall be long enough to permit ample room for tows going through, and that this length, combined with the elevation of the sill, shall be such that there will be only a very slight fall or " swell-head " there when the navigable depth on it is at a minimum. The last condition is an important one, as, if there is a fall of more than a few inches the current will prove too strong for ascending boats.

To illustrate the methods used in solving the problem an example from actual practice (Dam No. 3, Big Sandy River, West Virginia and Kentucky) is given. In this case the pass was assumed as 130 feet long and the " swell-head " was assumed as 0.5 ft. The lock, which was assumed to be opened during floods, thus assisting in the discharge, was 52 feet wide. After construction the actual swell-head, assumed in the calculations as 0.5 ft. with the lock gates open, was found to be about 0.4 ft. with the lock gates closed.

The following calculations were made to determine the principal dimensions: *

" Assuming a value of 0.5 ft. for the swell-head, the question then is to determine the maximum stage of the river, the discharge corresponding to which will pass through the pass and lock without causing an increase in depth of water above the dam over that below the dam exceeding 0.5 ft. This will give the elevation of the sill of the weir above the sill of the pass.

* Annual Report Chief of Engineers, U. S. A., 1897, p. 2538.

"To determine this the Chanoine formula is used, to wit

$$Q=M(LH+L'H')\sqrt{2g(Z+h)},$$

in which Q = discharge of river in cubic feet per second;
M = constant depending on stage;
L = length of pass in feet;
L' = length of weir in feet;
H = height of water below dam above sill of pass = stage in this case;
H' = height of water below dam above sill of weir;
v = observed velocity of approach;
h = height due to observed velocity of approach $=\frac{v^2}{2g}$;
$g=32.2$ feet;
Z = swell-head, as explained above.

"For any stage H the discharge through the pass will be $Q'=M(130H)\sqrt{2g(Z+h)}$, and through the lock $Q''=M[52(H-1.25)]\sqrt{2g(Z+h)}$, as the upper miter-sill is 1.25 feet higher than the sill of the pass; so that for the pass and lock $Q=Q'+Q''=M(182H-65)\sqrt{2g(Z+h)}$. For $H=5$ feet, $M=.703$, $v=2.72$ feet, and the equation becomes $Q=.703\times845\times8.02\times.784=3735$ cubic feet, which is considerably greater than the discharge corresponding to a 5-ft. stage. Making H in the formula equal to 6 feet, M becomes equal to .71, $v=2.99$ feet, and there results $Q=.71\times1027\times8.02\times.8=4678$, for the discharge by the lock and pass with a swell-head of 0.5 ft. If Z is taken as equal to 0.5 ft., and the sill of the weir is made 6 feet above that of the pass, H becomes 0.5 ft., and M is reduced accordingly. Taking L' equal to 140 feet, the discharge over the weir becomes equal to 180 cubic feet per second, so that the quantity of water that can pass the dam per second, the lock gates being open, without producing a greater swell-head than 0.5 ft., is 4858 cubic feet. The discharge of the river corresponding to a 6-ft. stage is 4910 cubic feet per second, so that the swell-head corresponding to this discharge would be but little in excess of 0.5 ft.

"In determining the length of the weir the following conditions must be satisfied: 1st. The area of discharge afforded by the weir must be sufficient, when taken in connection with those of the pass and lock, to permit the passage of discharges corresponding to all stages up to the level of the top of pier and abutment, without causing a greater swell-head than 0.5 ft. 2d. The discharge area of the weir should be sufficient to pass all discharges corresponding to stages up to that at which the natural river is navigable, without the removal of any needles from the pass. It may be stated, however, that the second condition will be satisfied by a length of weir that will satisfy the first.

"To determine the length of the weir, the elevation of its sill being fixed, the formula of Chanoine, modified to take into account the discharge through the lock, is used, to wit:

"$Q = M[LH + 52(H - 1.25) + L'H']\sqrt{2g(Z+h)}$, or substituting for L its value already determined, 130 feet, the formula becomes $Q = M(182H - 65 + L'H')\sqrt{2g(Z+h)}$; making H equal to 7 feet, H' becomes equal to 1 foot, and the formula becomes $Q = M(1209 + L')\sqrt{2g(Z+h)}$. For a stage of 7 feet, $Q = 6362$ cubic feet, $M = .73$, $h = \frac{v^2}{2g} = \frac{(3.21)^2}{2g} = .160$ and $(Z+h)^{\frac{1}{2}} = .812$, whence L' becomes equal to 129.3 feet.

"For high stages, the formula of Chanoine and De Lagréné is used, to wit: $Z = 1.5V^2\left(\frac{S^2}{S'^2} - 1\right)\frac{1}{2g}$, in which Z = swell-head, V = mean velocity before construction of works, S = discharge area of river before construction of works, S' = discharge area after construction of works $= LH + L'H'$, and 1.5 is a constant for cases where the lock gates are open. Transforming the above equation and substituting for $2g$ and Z their values 64.3 and .5, respectively, there results $S' = \left(\frac{1.5V^2S^2}{32.2 + 1.5V^2}\right)^{\frac{1}{2}}$. For the highest stage observed $H = 15.45$ feet, $H' = 9.45$ feet, $V = 4.7$ feet, and $S = 4576$ square feet, and $S' = LH + L'H' = 3260$ square feet, $= 2008.5$ square feet $+ 9.45L'$, whence $L' = 132.4$ feet.

"As the stage that would just cover the pier and abutment is 16.5 feet, it is thought best to fix the length of the weir at 140 feet, particularly as the rises during the summer are generally quite sudden.

"With the sill of the pass at low-water level, and 130 feet long; the sill of the weir 6 feet above that of the pass, and 140 feet long; for the 8-ft. stage Z becomes equal to 0.53 ft., for the 9-ft. stage it is less than 0.5 ft., and when the pier and abutment are submerged Z will not exceed 0.5 ft. appreciably."

Foundations and Details of Construction.—(See also pp. 372 and 518 and after.) The foundation for a movable dam should be preferably of masonry, without any timber except where it would be protected from seepage and currents or could be easily replaced, and wherever practicable should be on bed-rock; where this is not feasible substantial aprons or other protection must be provided for the downstream face. This type of dam, owing to the regulation of the pool, is more subject to the dangers of currents and reactions than a fixed dam, and should any settlement or undermining occur it may throw the superstructure out of order. For this reason the substructure should be made secure and of permanent materials, especially as repairs may necessitate expensive cofferdams and a long interruption of navigation. A fixed dam will safely stand a settlement which would be dangerous to a movable dam, and can, moreover, be more easily repaired.

The long experience of European engineers in this field has shown the wisdom of using nothing but permanent foundations for movable dams, and they discarded

long since the use of timber where it would be exposed to wearing from the currents. If timber is intended to last permanently in submersion, it must be protected from flowing water or it will be gradually worn away. Thus the sheathing on a dam subject to a constant overflow wears rapidly, and we have removed oak plank from such positions which had been worn from 3 inches to less than an inch in thickness in twelve years. (See also "Materials of Construction, p. 384.) The designer should always bear in mind that the fewer the repairs to be made the better, and these should be rendered as easy as possible.

While rock is the best foundation for movable dams, as it removes the dangers of undermining from above and from below, comparatively few dams have had this advantage. The great movable dams of the Ohio, exposed to rapid floods and strong currents although not of great lift, are almost all founded on gravel; (see p. 595 and accompanying plates in Chapter VII); the bridge dam, at Mirowitz on the Moldau, with a lift of $12\frac{3}{4}$ feet and $16\frac{1}{2}$ feet on the pass sill, is on similar material; the Suresnes dam near Paris, with 10.7 feet lift and 15.0 feet on the pass sill, is on clay; one of the Oise bridge dams is on sand; the Nile dams of Assiout and Zifta are on silt; and most of the Mohawk River bridge dams with lifts of 8 to 15 feet are on materials varying from a sandy gravel to clay. (See "Mohawk River" in tables at end of book.) Where not on rock, a line of tight sheet-piling should be driven along the upstream face of the foundation and be continued well into the bank. The tops should be embedded in the masonry, as it has been found that if they are not thus covered the water will leak down through the joints. This line is especially important on sandy foundations, as if there is seepage underneath it will gradually carry out the particles and may undermine part of the dam, as happened to one of the bridge dams on the Oise. (See p. 374.) To guard against this the German engineers use fascines (Fig. 211, p. 579), and the British engineers a bed of gravel (Pl. 73, Fig. 2) so as to prevent the escape of the particles. (See also p. 373 and after.) Where the foundation is on gravel this danger appears to be slight, judging from examples where insufficient or no sheet-piling has been used; the water undoubtedly percolates beneath the dam, but cannot remove the gravel, and thus no undermining takes place. The particles of sand in the gravel at the outlets of the leaks are washed out, but their volume is negligible, and while there is left a clear channel through the interstices of the pebbles no further change appears to occur.

The downstream side is as vulnerable as the upstream, being exposed to the strong rush and eddying of the escaping water, which carries off all loose material. To protect a vulnerable foundation, heavy riprap or an apron, or both in combination are necessary, as a few maneuvers of the dam may suffice to scour holes, if in gravel, 30 feet and more in depth, and the function of the protection is to carry the reaction to a point downstream where the violence can spend itself at a safe dis-

tance from the dam. At the first wicket dam built in America, on the Ohio just below Pittsburg, the foundation was narrow and much trouble was caused for several years by the scour, the reaction washing large holes in the gravel bed, and carrying out the comparatively light riprap protection. The size of the latter was finally increased to pieces of about 1 cubic yard or over; piles were driven in it, and it was extended to a distance of about 50 feet downstream of the foundation proper, after which it gave little trouble. Similarly on the Suresnes dam on the Seine, with a narrow masonry foundation, the riprap protection had to be increased from pieces of about 4 cubic feet to pieces of about $1\frac{1}{4}$ cubic yards before it could withstand the rush of water.

A very effective and economical protection was introduced on the Ohio dams about 1900, consisting of a crib 20 feet wide and with an upward slope of 2 feet placed against the downstream face of the foundation. (See Pl. 54.) Below this is dumped roughly a pile of riprap 15 to 20 feet wide, of stones measuring from $\frac{1}{2}$ to $1\frac{1}{2}$ cubic yards apiece, and with the exposed face on a slope of 1 vertical to $1\frac{1}{2}$ or 2 horizontal. This angle of slope is important, since when made flatter it was found that the water tended to undermine or carry away the pieces, as they apparently projected too far into the rush of the sub-surface reaction. With these arrangements it has been found that the worst scour occurs about 150 feet below the dam, at an ample distance from all points of danger, the upslope of the apron apparently throwing the reaction far downstream.

Usually little or no riprap protection is placed above the foundations in American practice, as it does not seem to have been required with the types of movable dam generally employed, although with the bear-trap weirs of the Ohio holes above the dam have been occasionally scoured out to a depth of 16 feet and over, requiring the use of a good protection of riprap. In European practice, on the other hand, it is customary to use a considerable amount. On the Nile dams founded on silt, this was done largely to keep infiltration from reaching the structure, a clay blanket being used underneath. (See Pl. 73, Fig. 2.) Another point of difference is that European engineers frequently use a line of sheet-piling along both the upstream and the downstream sides of the foundation; in America it is usually considered to be a better practice to employ none on the downstream side, but to let all seepage escape freely and thus minimize the danger of a concentrated upthrust on the foundation. (See also p. 367.)

If the masonry is of cut stone the two upper courses should be doweled together, and in many cases long bolts are run horizontally from near the upstream to near the downstream face. These precautions are necessary to prevent any of the pieces being loosened by blows from drift, ice, etc. Where the foundation is of concrete of small thickness—say 4 or 5 feet—it is a good plan to tie it together with reinforcing rods of $\frac{3}{4}$ or 1 inch square section, spaced 3 to 5 feet apart.

The sills supporting the bottom of the curtain of the dam (Figs. 207 and 208) should be made preferably of cast iron, although where always under water wood has been generally employed in the United States and occasionally wrought steel. Cast iron is, however, much preferable, since the wood becomes gradually worn away by the currents and can only be replaced with difficulty and expense, besides being liable to a splintering of the edges from drift, etc., dragging over it. On the dam at Créteil on the Marne a sill of plates and angles was used, and on the wicket weirs of the Big Sandy dams is to be found a similar design. (See Pl. 59.) In the latter cases there was found to be a great advantage in setting the wrought sills as compared with cast ones; owing to their being in lengths up to about 32 feet, they gave a very exact alignment for the wickets and provided a stiff connection for them. Cast sills cannot well be made more than 2 wickets in length, or about 8 feet, owing to their liability to warp when cooling after being cast, so that 4 such pieces would have to be set where one wrought piece could be used. There is also more or less unavoidable

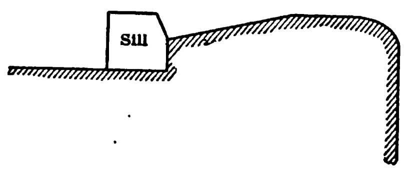


FIG. 207.

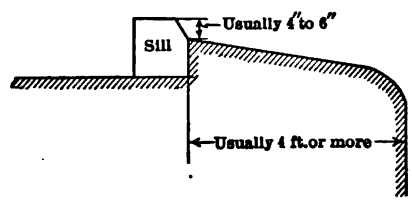


FIG. 208.

irregularity of outline in such castings, but they have the great merit of resisting rust much better than the steel.

The sill should be well anchored down, as it has to stand many blows and much hard service. With high-lift needle or wicket dams, where the sill carries unusual pressure, it is generally necessary to use both vertical and horizontal anchor bolts (see Pl. 59). For ordinary cases, however, practice has shown that stout vertical bolts are sufficient.

The depth behind the sill, or the recess, should be sufficient to allow all movable parts to lie down, with an inch or two of clearance above them, so that any passing object would strike the sill rather than the parts below. The depth of these recesses varies from 15 to 20 inches or more, the Boulé dam at Libschitz on the Moldau having a depth of $3\frac{1}{4}$ feet. It should, however, always be made as small as possible, since it acts as a catch-basin for gravel, sunken drift, etc.

The height of the sill above the masonry on the upper side is usually from 4 to 6 inches, as may be required to support the ends of the needles or wickets. It is made small to prevent gravel and loose stone catching against the sill when the dam is down, and which would interfere with the placing of the needles or wickets.

The upstream surface of the foundation masonry has generally been made of the same elevation as the top of the sill, under the idea that it would act as a guard to the sill (Fig. 207). In more recent practice, however, such a guard has been considered unnecessary, and the masonry has been made to slope up towards the sill, with the view of eliminating the pocket and giving less chance for gravel to stay in it. (Fig. 208.)

In the coping of the masonry there should be provided sockets into which uprights can be placed for use in coffering-off any portion of the dam for repairs to the superstructure. These can be made 9 to 12 inches deep and 8 inches square or more at the top, tapering towards the bottom so the uprights can be pulled out easily. A common form is a piece of 6-inch pipe set in the masonry.

Drift-chute or Regulating Weir.—It is very desirable, particularly in high dams, to provide a regulating weir whose closing apparatus shall always be under easy control, and which can be maneuvered with certainty, night or day, by the watchman. The proper location for such a weir is away from the lock, so that the rush of water passing through it will not disturb navigation. It is usually placed next to the abutment, and is separated from the main dam by a masonry pier, but the choice of location depends somewhat on which side the drift runs during floods, as the weir should be placed near that side if it is to safeguard the dam. It should be of sufficient length to pass safely all the drift up to the stage of water at which the dam is lowered, and to assist in discharging the surplus water at medium stages, but it should not be of such dimensions as to render its maneuvers uncertain. On wide rivers, as the Ohio, two or more such weirs (usually known as "drift chutes") are used. A description of these will be found in Chapter IX.

Abutment and Protection of Banks.—The banks below the abutment and the lock must be protected as described in the last chapter, since they are similarly exposed to washing, and the design and construction of the abutment should be based on similar principles. (See p. 524 and after.)

Leakage of Movable Dams.—This question was formerly considered of great moment in comparing the relative advantages of fixed and movable dams, the former being supposed to be almost watertight. In point of fact, however, the fixed dam if of timber becomes very leaky in the course of time, as the water enlarges every crack or hole in the woodwork which it can pass through. The movable dam, on the other hand, has proved very amenable to being made watertight, and with a little work will keep the pools full even in the driest seasons. This is secured in the case of wickets by placing pieces of timber in the spaces between the wickets; with gates or curtains vertical planks are used over the joints at each upright or trestle; while needles can usually be placed close enough to require no such assistance. Moreover there is usually a considerable amount of submerged leaves, grass, etc., which the water carries into the joints, and if this

proves insufficient, cinders, fine gravel, sawdust, etc., can be sifted down against the upstream side of the dam, and this will effectually close all the remaining spaces. It is not uncommon to see portions of the downstream face become dry in a few days after this last method has been employed. Some of the steel Chanoine wickets of recent design have bent plates fastened to their edges so as to reduce the leakage space, as shown by Fig. 209*a*, p. 571.

Ice and Movable Dams.*—The problem of ice is a serious one with movable dams, as when severe frost begins the ordinary types usually have to be opened lest the various parts become frozen together. Even if the season is not severe enough to produce this danger, and only a moderate amount of ice forms in the river's basin, the floods will carry it into the stream, and if the dams are still partly in place the drifting pieces will catch in the trestles and other parts, and may pile up until lowering becomes impossible, and serious damage may ensue. Because of this danger, movable dams on the northern rivers of the United States are almost always lowered at the approach of winter and kept down until the spring. Should the season be one of low water, navigation has to stop, the dam being then of no benefit. In certain cases, as where the dam is just below a city, additional inconveniences result, because all harbor navigation comes to a standstill, and factories which are dependent on the river for coal or other supplies are put to much expense in procuring them elsewhere. This state of affairs has happened more than once with Dam No. 1 on the Ohio, just below Pittsburg, and on certain occasions, at urgent instance of the manufacturing interests, the wickets have been raised or have been kept in place in winter-time in order to provide sufficient depth for harbor navigation. The result, however, has usually been disastrous, as ice and high water coming suddenly have more than once wrought havoc with the dam. In one or two instances the pass wickets could only be lowered by pushing them upstream with a steamboat until the ends of the props cleared their supports and allowed the wickets to fall, usually in a somewhat damaged condition.

The bridge dam and the rolling dam have proved better able to cope with ice than other types so far employed, but their application is limited because of financial reasons, and thus they cannot well be used for large rivers of high floods where navigation requires wide openings and good head room. The dams on the lower Seine and on the Oise (see pp, 528, 626 and 632) are usually kept in place nearly all winter, while rolling dams elsewhere have successfully stood severe tests with ice. It is believed also that the use of wide-span trestles, as described and shown on pp. 568 and 572, may simplify conditions, since there are few pieces among which floating ice can lodge. On the Upper Seine, where the weirs are of needles and the passes of wickets, it is customary to regulate the pool by lowering part of the latter when ice begins to run and to keep the weir closed as long as possible. The ice is thus

* See also various reports, International Congress of Navigation, 1908.

kept away from the closely-spaced pieces of the weir, and has an unobstructed way of escape through the pass.

A severe test with ice occurred on the Seine in 1894. The weather turned cold very suddenly, and the dam-tenders all opened their dams as hastily as possible, with the result that an artificial flood carrying a vast amount of ice was produced. This flood of course became more severe as it passed from pool to pool, the first dam to suffer being the needle dam of Courbeton above Paris, which was caught and wrecked by the ice. The needle dam of Bezons on the Lower Seine was the next to be endangered, and it was only by chopping at the downstream sides of the needles until they broke and let the ice pass that any of the dam was saved. The trestles were lowered in any way possible, and many of them were torn loose, while the greater portion of the needles was lost or destroyed. The same flood practically destroyed the movable portions of the Martot needle dam, the one farthest down the river, and the bridge dams were also endangered, but they withstood with little injury ice-gorges similar to those which had just wrecked the needle dams above. Thus at the Poses dam (described on p. 626 and after), the ice formed in less than 10 minutes a gorge across one of the passes, extending in a solid mass from pool to foundation, the cakes being about a foot thick and standing on end against the curtains and uprights. A hole was quickly chopped to a depth of about 14 feet in front of three curtains which were already partly raised, and they were then rolled up and one frame was pulled upstream, leaving a clear opening about 7 feet, through which the current gradually carried the ice (assisted by the dam tenders) until the remainder of the pass could be freed.

An additional danger from ice, alluded to before, occurs when the discharge is small, as the spray freezes the parts together, or coats them with ice until the masses prevent the lowering of the dam. DeMas quotes an example of the first-named difficulty which happened on the Meuse at Verdun in 1875, when sudden cold froze together the needles of the dam in a few hours' time. The river was rising and began to inundate the adjoining town, but before much damage occurred the increased head of water broke through part of the dam and afforded relief. Because of such dangers it is customary in Europe to lower movable dams whenever the thermometer falls to about 22° F.

In the year 1906 an international competition was opened under the auspices of the Austrian Government in order to secure a suitable type of movable dam for the canalization of the Upper Elbe in Bohemia. A type was desired which could be used all winter in order to produce water power and which would not be injured or blocked by ice. The width of opening was not to exceed about 75 feet. A considerable number of designs were submitted, most of which resembled in general principle the Taintor gate type (see Fig. 202, p. 524), and the ones accepted were of this

general design with counterbalancing arms, the travel of the gate being such that it could be lifted above high floods.

Splashes from Movable Dams.—The old-time method of navigation by "splashes" or "flashes," in which the water was stored in a pool closed by a few needles or stop-planks and then suddenly released so that boats could float down on the artificial flood (see p. 537), has found a modern application in the wicket dams of the Kanawha River. This stream flows into the Ohio about 250 miles below Pittsburg, and exports a large amount of coal to Cincinnati and other points further down. As this deep-draft navigation depends on floods to pass the unimproved portions of the channel, it can take advantage of small rises if assisted by the release of some of the water in the Kanawha pools and thus reach the destination safely. It has therefore occurred several times, beginning with the year 1900, that when a rise of a few feet was coming down the Ohio, the coal fleets have waited just inside the mouth of the Kanawha, and at a suitable time some of the water has been released from the pools above and the combination of the artificial and the natural flood has enabled the barges to pass out and reach their destinations. The average velocity of the flood crest on the Kanawha over the 54 miles from which the water is released has been found to be about six miles per hour and a rise of $6\frac{1}{2}$ feet in the Ohio has been increased by these means to $8\frac{3}{4}$ feet.* (See also p. 596.)

View of Weir of Dam No. 6, Kanawha River, W. Va., Illustrating the Effect of Drift on Trestles of Movable Dams.

General Remarks. (See also "General Remarks," p. 530.)—Movable dams, although solving in principle the problem of slack-watering rivers without changing their natural regimen, and now employed almost universally for that reason on European streams,

* See "Engineering Record," Jan. 28, 1911, for further details. The velocity of flood crests on the Mississippi has been given on p. 34.

are open to several objections in comparison with fixed dams, the chief of which are greater expense in establishment, operation, and maintenance; much more danger to employees when operating (although in actual experience accidents have been very rare); and greater liability to injury from drift and ice, especially on American rivers. The problem of drift has been met thus far without serious injury by constant watchfulness on the part of the dam-tenders, and as the districts along the rivers become settled and the timber cleared away, this danger should gradually decrease. The problem of ice, however, referred to in the preceding paragraphs, will always remain a serious one, for while extreme cold will stop navigation by freezing over the river, there are usually many times when boats could continue to run in less severe weather, provided there was a sufficient depth of water. It is at such times that the movable dam may be found wanting, and while certain types have in a large measure proved reliable at all times, they have been applied heretofore only to comparatively short spans or narrow rivers, and do not seem fitted, on account of costliness or lack of head room, for the closure of wide streams unless in exceptional cases.

In September, 1911, the Ohio River Dams (see p. 595) just below Pittsburgh were subjected to a severe test from high water and drift. Excessive rainfall upon the headwaters caused a sudden flood which was accompanied by immense quantities of drift. Lowering the dams was commenced as soon as possible, but the drift accumulated so fast that operations began to be seriously hampered. Dams 3 and 4 were successfully lowered, but at Dam 5, after lowering 260 feet of the pass, the maneuvering boat (no service bridges are used on the passes of the Ohio dams) was forced over the structure by the gathering mass of drift, and operations had to be suspended. At Dam 2 the drift stopped the lowering when less than half of the pass wickets had been bedded. An attempt was made to lower additional wickets by pushing a barge against them from below, but without success, and no further efforts could be made. The rise of the water finally caused the wickets to swing when the water reached a depth of 2.6 feet above their crests. Prior to that time the head at the dam had been 8 or $8\frac{1}{2}$ feet; this was reduced to $3\frac{1}{2}$ feet by the swinging of the wickets, of which only about 15 per cent remained upright. The river continued to rise, however, until it reached about 6 feet above the level of the crest, and a solid dam of drift blocked the entire length (700 feet) of the pass. As soon as the water receded sufficiently navigation was passed through the lock, and within a week all the drift had been removed and the wickets had been lowered. The damage was remarkably small; of some 175 wickets only 13 were disabled, while only 21 props and 10 horses were broken or bent. Much of this damage occurred from some barges which broke loose and rammed the dam. The injuries to the other dams were proportionately small.* (See also p. 564, at bottom.)

* See Annual Report, Chief of Engineers, U. S. Army, 1912, p. 2328, for further details.

A movable dam is rarely desirable except where property might be injured by the higher flood levels of a fixed dam, as it is a constant source of anxiety and expense. On the other hand, because of the shorter length required to pass a given flood, its first cost in some cases may prove not greatly in excess of that of a fixed dam, and owing to the possibility of keeping down the levels of moderate floods a higher pool elevation can often be used than would be possible with a non-adjustable fixed dam. At suitable localities a compromise between a fixed and a movable dam may be advantageously secured by employing a fixed dam with a movable crest, as described on p. 521 and after.

Time of Operation of Movable Dams.—In the operation of a movable dam the question of time is usually important only in the lowering, since the rise of a flood is generally much swifter than its falling, and, in addition, drift and débris run chiefly on a rising river, and unless a free outlet can be afforded quickly the pieces may get caught in some of the movable portions and prevent the dam from being lowered. It has happened in several instances, both on American and European rivers, that part of a dam has been destroyed from this cause. By the time the river has begun to fall such floating objects have usually gone by, and since the water falls comparatively slowly there is generally ample time to put the dam in place before the river has fallen to an extent which may hinder navigation.

The following paragraphs give the time required to maneuver various types of movable dams.

Needle Dams.—The Andrésy dam on the lower Seine, with a 9-ft. lift and an opening of 380 feet, took about $8\frac{1}{2}$ hours for the opening and required the labor of eight men. To close the dam the same number of men were required, and operations lasted about 10 hours.

The original Louisa Dam (see p. 563), 270 feet in net length, took about 4 hours to put in place, under ordinary circumstances, and could be removed in 2 hours under emergency, but usually was removed much more slowly. At Dams 1 and 2 on the same river the Chanoine wicket weirs can be lowered in less than an hour by using the operating boat, and the deep needle passes are usually lowered in less than 2 hours. By using the tripping bars provided on the weirs the lowering can be accomplished with great rapidity, and it is probable that both pass and weir, of a total length of 310 feet, could be lowered in 1 hour's time were it advisable or necessary. The regular employees are three in number at each dam, and two or three more are temporarily employed for complete maneuvers.

Chanoine Wicket Dams.—To lower the pass of La Mulatière Dam on the Saône, comprising a length of 340 feet and with a low-water lift of 11 feet 10 inches, took $4\frac{1}{2}$ hours under the most favorable conditions, and to raise it took 8 hours. This portion of the dam is operated by a steam engine traveling on a trestle bridge. (See p. 595.)

The lowering of the pass and weir of one of the dams on the Kanawha River (see p. 595), comprising a total length of 540 feet and with a low-water lift of about 8 feet, has been done in about $4\frac{1}{2}$ hours under the most favorable conditions, and the raising in about 9 hours. The more usual times taken for lowering and raising are about 8 and 12 hours respectively. The average time taken for raising one of the weir trestles, including attaching the floor and rails and moving the winch, is about $3\frac{1}{2}$ minutes. At Dam No. 11 a record has been made of lowering 11 wickets in a few minutes by using a tripping spar from the bridge to pull the heads of the wickets upstream until the props became unseated. Another pass 304 feet long has been lowered in about half an hour. The head on the pass wickets when being lowered usually varies from 2 to $4\frac{1}{2}$ feet, and from four to six men are needed for operations.

Dam No. 6 on the Ohio River, about 30 miles below Pittsburg, comprising a length of nearly a thousand feet, under the most favorable conditions has been raised in about $2\frac{1}{2}$ hours and lowered in a little over 1 hour. The 150 wickets of the pass, covering a length of 600 feet, have been raised in 1 hour and 40 minutes, with a stage of $6\frac{1}{2}$ feet in the open river. (See p. 595 and accompanying illustrations.)

The speedier maneuvers of American dams as compared with those of Europe are necessitated by the quicker rates of flood rise, information about which has been given on p. 35.

Trestle and Curtain Dams.—Villez Dam, Lower Seine. This dam has a maximum depth of 13.1 feet on the pass sill. To open the two passes and the one weir takes from 22 to 38 hours, depending on the rate of rise of the flood. The length of openings is 660 feet, and the 22-hour operation is equivalent to removing 30 feet per hour.

Suresnes Dam, Lower Seine. (See p. 618 and after.) This dam has a depth of 15 feet on the sill of the deepest navigation pass, and a lift of 10 feet 8 inches. To lower the 57 trestles of the pass takes about 3 hours, or 3 minutes per trestle with six to seven men; to raise them, about 5 hours, or 5 minutes per trestle. This time is for the trestles only, and does not include handling the gates or curtains.

Bridge Dams.—Mirowitz Dam, Moldau. (See p. 628 and after.) To raise one pair of uprights by hand power took 20 minutes; to lower it, $14\frac{1}{2}$ minutes. When the electric winch was used the time became $1\frac{1}{2}$ and $1\frac{1}{3}$ minutes respectively. To raise one gate by hand power took 45 minutes; to lower it, $2\frac{1}{6}$ minutes, using a brake; with the electric winch, $1\frac{3}{4}$ minutes for each operation. These figures were determined by special experiment, and do not include loss of time for moving and setting the winch, etc.

Poses Dam, Lower Seine. (See p. 626 and after.) To raise one set of uprights of the navigable passes by hand power takes 30 minutes; to lower it, 11 minutes. With a steam winch, the raising took 14 minutes, and the lowering 10 minutes. These

periods are shortened when the electric winch is used. The foregoing does not include the time necessary to maneuver the curtain.

To operate the 91 pairs of uprights, covering about 692 lineal feet of dam, etc., but not including any maneuvers of the curtains, took the following time :

Hand operation (used originally) with one winch:	Raising,	60	hours with	20	men
	Lowering,	40	"	14	"
Steam operation with one winch:	Raising,	18	"	7	"
	Lowering,	7	"	6	"
Operation with one electric and one steam winch:	Raising,	8	"	12	"
	Lowering,	7	"	5	"

The electric winches are now used almost entirely for operating this dam. With one of them one man can raise one of the longest pass curtains, which must be handled very carefully, in 9 minutes. With hand operation this takes four men 15 minutes.

It may be mentioned that a type of dam in which the pieces are lowered in order to open the river, as with a Chanoine wicket dam, is naturally more speedy in operation than one in which they have to be raised, as with a bridge dam.

Dams on the Lower Seine.—This system of movable dams, dimensions, etc., of which are given in the tables of locks and dams at the end of the book, was completed about 1885 and provides a least channel depth of $10\frac{1}{2}$ feet between Paris and the sea. For a year of average floods full maneuvers of the dams are required only twice, and the dams are then out of position for a total of about 50 days. For about 12 days of this time navigation has to stop owing partly to lack of head-room and partly to dangerous currents at the bridges. Downstream navigation has of course to stop before the upstream, owing to the greater difficulty of handling the boats. With the river fully open the current reaches a maximum velocity of 5 feet per second, except at the bridges, where the maximum is $7\frac{1}{2}$ feet. With the river partly open these velocities become $1\frac{3}{4}$ and 3 feet per second respectively. The dredging required for the maintenance of the channel has been described on page 117.

CHAPTER VI.

NEEDLE DAMS.

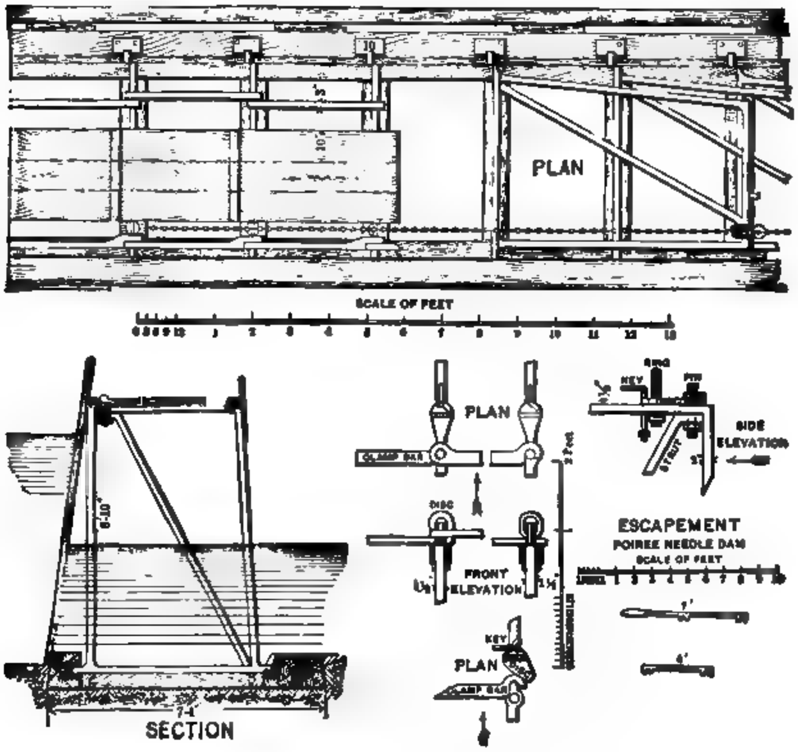
History.—Although the bear-trap was in temporary use on the Lehigh River at an early date, the pioneer of movable dams on navigable rivers was the needle dam, invented by M. Poirée in 1834, and first constructed at Basseville, France. It is called a needle dam because the wall or "curtain" which holds and supports the water is made of needles or wooden spars ranged side by side across the river.

Modern needle dams are constructed on the same general principles as this first dam. The curtain of the dam is formed of the needles, which are of a size suited to the head of water, but rarely larger (except in America) than one man can control. Their bottoms rest against a sill, and their tops against bars known as escape-bars or support-bars, which are in turn supported by trestles turning on hinges fastened on the floor. The trestles are lowered behind the sill when not in use. A walk-way, hinged to the trestles, or made of loose planks, provides access for maneuvering when the dam is up. (See cuts on pp. 560 and 562.)

In the Basseville dam the trestles were placed $3\frac{1}{4}$ feet (one meter) apart, and weighed 242 pounds, with a height of $6\frac{1}{4}$ feet, the lift being only $3\frac{1}{4}$ feet. The distance between the trestles of European dams is usually made about 4 feet, but on rivers in the United States the distance has been increased to 8 and 10 feet, and on the passes of the dams on the Big Sandy River, described further on, the trestles are 20 feet apart. This increase of spacing has been found of much advantage, as it reduces the number of moving parts and gives more freedom for the passage of drift. The first navigation dam in Europe in which the trestle spacing was increased from the customary $1\frac{1}{4}$ meters was that at Wegstädtl on the Bohemian Elbe, completed about 1907. Here a spacing of 9 feet 9 inches centers was used, and the example has been followed on neighboring dams.

Maneuvers of Trestles.—All the trestles are connected, either by chains made in separate lengths and fastened to the heads of adjacent trestles, or by one long chain running through the heads, and fastened to them at the proper intervals by clamps or by latches, and worked by a winch on the masonry. The latter method is one almost universally adopted in the modern needle dam, and by its means two to six trestles can be lifted at the same time. (See illustration of the Louisa dam on p. 565.) When it is desired to raise the dam the first trestle is hoisted up and connected to the masonry by the escape-bar and by the floor, and a similar

process is followed with the succeeding trestles until all are in place. Where a continuous chain is used it is released from each trestle in turn, as the latter becomes vertical, by a man on the foot-bridge; where separate chains are used a portable winch usually is carried from one trestle to the next. This method, how-



General Drawing of an Early Poirée (Needle) Dam.

ever, is rarely used except for very small trestles, the continuous chain having been found more convenient. It was used for a while on the Klecan needle dam on the Moldau (see illustration on p. 581), but was abandoned later in favor of a single chain. After all the framework is in place the needles are put in, sometimes from a boat, but usually from the foot-bridge.

To lower the dam, the reverse operation is pursued, the needles being first removed and the trestles lowered in order.

Placing and Removing Needles.—Small needles are usually placed and removed by hand. To place them, the head is held against the escape- or support-bar, and the end plunged upstream in the current which draws it round till it strikes the sill. In the Marne dams, where the needles are $4\frac{3}{4}$ inches square and weigh up to 100 pounds, the same method is used; but the downstream sides are provided with hooks which fit over round support-bars, thus holding the heads in the exact position desired. As the length from the hook to the end of the needle is made to exceed the length from the support-bar to the sill by $\frac{3}{8}$ to $\frac{3}{4}$ inch, the foot of the needle scrapes along the masonry just as the needle becomes upright. This takes away the shock from the support-bar almost completely, and assures the normal placing of the needle. In order to permit this maneuver the sill has to be sloped up as shown in Fig. 208, p. 550.

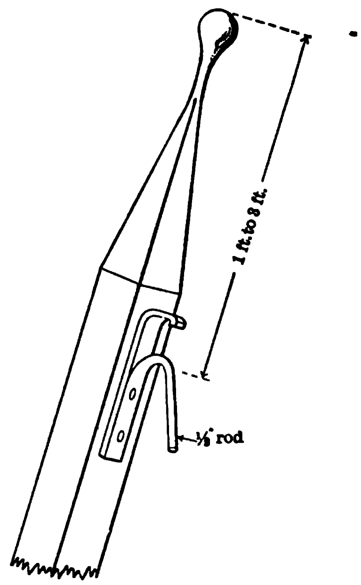


FIG. 209.—Hook Needle.

The removal of these "hook needles" (Fig. 209) is sometimes effected by a winch carried on a truck, but ordinarily by a plain wooden lever, which raises the needle till its foot passes over the sill, when the needle swings free on the support-bar and in the current. It is then lifted up by hand from the footway and loaded on to a car. Another method of removal, and a much earlier one, is that known as escaping. The escape-bar, which in all earlier dams was a loose bar connected when in use to the trestles by claw ends or by bolts, was later on hinged at one end so as to swing horizontally, the other end resting against a disc or a cranked post shaped so that on being turned it released the end of the bar and allowed it to swing. (See cut on p. 560.) The force of the water then carried the needles (each bay of which was first tied loosely to a line) through the opening and they were picked up below. This is known as the Kummer escapement, from the name of its originator, and the method was largely adopted in Belgium and elsewhere. However, fixed bars with hook needles have supplanted it in nearly all recent dams, as the needles are more easily and safely handled, and are not injured by being flung against the trestles, as happens when they are escaped. Experience has shown, moreover, that the need for a sudden removal of the entire dam such as the Kummer method renders feasible, rarely if ever occurs.

A hook-needle dam, 126 feet long with 9 feet on the sill and a 6-foot lift, was built on the Mohawk River, N. Y. The needles were $5\frac{3}{4}$ inches square and about

13 feet long, but the operators appeared unable to handle them, and the dam was finally replaced by the gate type.

The removal of a plain needle by hand is always attended by some risk, because as soon as the needle clears the sill the current snatches it downstream, and the operator may be pulled into the water. At the Ile Barbe dam at Lyons a home-made contrivance is in use for avoiding this danger; it consists of an iron U-clip tied to about 5 feet of line, the other end of which is attached to a light

Section of Needle Dam on the Lower Seine Prior to 1880.

wooden lever on a portable stand. To remove a needle, the clip is put round the head and a quick jerk on the lever pulls the needle clear of the sill, and the line then holds it until it can be taken out of the water. On some of the irrigation canals of the Nile Valley, where the water was regulated by needles, the sluice-tenders became very expert in the maneuvers, and by taking advantage of the flotation could handle needles 9 inches by 6 inches by 30 feet long in 25 feet of water.* These have been largely replaced, however, by horizontal beams or by steel gates.

* Proceedings Int. Eng. Congress, Glasgow, 1901.

Another method, which has been in satisfactory use for some years on the dams of the Big Sandy River, and which does not bruise the needles, as often happens in the method of escapement, is to attach a long chain or rope to their upstream sides and pull them away with a crab or an engine.

Regulation of the pool with hook needles is obtained by pulling up the needles till they swing clear of the sill and rest on their hooks in the current. With plain needles some of the pieces are either removed entirely or the heads of alternate needles are pushed upstream a few inches with a lever and then supported by wooden blocks, allowing the water to escape around the sides. The latter method will discharge a large amount of water.

Needle Dams in the United States.—The first dam of this type in the United States was completed in 1896 at Louisa (now known as Dam No. 3) on the Big Sandy River, a tributary of the Ohio.* The pass was made 130 feet long, with 13 feet of water on the sill and trestles 4 feet apart, and the weir 140 feet long, with 7 feet of water on the sill and trestles also 4 feet apart. Later on the alternate weir trestles were removed, making the spacing 8 feet. After a service of about 10 years the trestles became greatly weakened by corrosion and the dam was rebuilt under designs practically similar to those adopted for Dams Nos. 1 and 2 (completed in 1904) on the same stream. As originally built, however, it comprised several special features which are worthy of note, as they had a considerable influence on the design of needle dams built afterwards. (See accompanying illustrations on pp. 565 and 572 and after, also Pl. 58.)

The adoption of needle dams for this river was due to the extremely small low-water discharge, which has fallen as low as 48 cubic feet per second.

As the river carries large quantities of sand it was desirable to have the recess behind the sills as shallow as possible, to avoid the accumulation of deposit over the trestles. The latter were accordingly shaped like an inverted V without any axle, the bracing, etc., being placed so that they lay down one inside the other, as shown in the cut of the weir trestles, instead of one on top of the other, as is the usual way. By this means a height of sill of only 15 inches was needed. They were all raised by a continuous chain worked from the masonry, and could be lifted or lowered, if necessary, six at one time. The chain rolled on pocket wheels in the heads, and could be attached to or released from them at any point by means of latches.

The needles, both for the pass and for the weir, were made of white pine, 12 inches wide, and weighed 263 and 80 pounds apiece, respectively. The former were about 14 feet long, $8\frac{1}{2}$ inches thick at the bottom and $4\frac{1}{2}$ inches at the top, being designed for a strain of 1200 pounds per square inch; the latter were about 8 feet

* See Annual Report of Chief of Engineers, U. S. A., 1897, p. 2530, and after, and Trans. Am. Soc. C. E., vol. xxxix, p. 460, for full description. See Pl. 46 for locations.

long, $3\frac{1}{2}$ inches thick at the bottom and $2\frac{1}{2}$ inches at the top. The width of 12 inches was adopted to save leakage, and proved very satisfactory, as with the extreme low-water discharge of 48 cubic feet per second, and a head of 12 feet, 2 inches on the pass, the total leakage through the dam was only 9 cubic feet per second.

Under a full load the needles showed a considerable deflection, but did not take any permanent set, and the only cause of breakages was the presence of knots or other defects in the timber. They were handled partly by men and partly by a small derrick-boat with an engine, and gave little trouble in maneuvering. Those on the weir were placed by hand, and if any had to be put in under a full head, as in regulating, their tops were held against the escape-bar and the ends plunged in the water as before described. In this case a rope was generally placed over the lower ends and used as a check against the force of the water. The pass needles were placed by the derrick-boat, and could be put in without great difficulty with a head of 3 or 4 feet. If the dam was raised during the dry season a hook attached to a line running to the engine on the maneuvering boat, or better still, a long wooden lever, was used to press the needles all together, thus closing any spaces. These general methods are still in use on the enlarged dam.

The regulation was done principally by *repoussing*, or holding upstream by wooden props about 12 inches long the heads of alternate needles, thus allowing the water to escape between. Next to the pier, on the weir side, a space of 12 feet was left, provided with two rows of needles, one above the other, supported by a lower and an upper escape-bar, and this also provides a convenient means of regulation and of passing débris. Several needles 6 inches wide were substituted later for 12-inch ones on the weir to facilitate regulation, being more easily removed. Lastly, in good weather, the water was allowed to flow over the tops of the needles to a depth of 6 or 8 inches, along the entire crest, thus reducing the work of maneuvering for small rises.

The removal of the needles was effected by attaching the upstream side of each one to a chain, allowing about 2 feet of slack between, and then pulling it from the boat, which was moored about 100 feet above the dam. By this means all the weir needles could be removed under a full head in a very short time, but the operation was always done slowly, so as to retain the pool at about the proper level. The trestles were also provided with swinging escape-bars, but as the needles became bruised against the ironwork in falling, owing to their unusual size, this method of release was abandoned.

The close spacing of the trestles (4 feet), which prevented the lowering of the others if one was blocked, eventually led to the destruction of much of the ironwork. A piece of drift became entangled in one trestle, stopping the maneuvers until it could be freed. As other drift was running, the pieces began to catch in the trestles

Raising the Weir Trestles.

Weir Trestles Lying Down.

VIEWS OF THE NEEDLE DAM (DAM No. 3, BIG SANDY RIVER) AT LOUISA, KY., U. S. A., AS ORIGINALLY BUILT (1896).

Placing the Pass Needles by Means of Experimental Revolving Shelves

View of Pass Trestles and Needles. (See Pl. 58.)

VIEWS OF NEEDLE DAM (DAM No. 3, BIG SANDY RIVER) AT LOUISA, KY., U. S. A., AS ORIGINALLY BUILT (1896).

General View of the Lock and Dam as originally Built (1896). (See p. 563.)

General View after Reconstruction (1908), Looking Upstream. (See p. 569 and illustrations on p. 568.) The higher Water Level on the Downstream Side is due to the Completion of Dam No. 2.

VIEWS OF THE NEEDLE DAM (DAM No. 3, BIG SANDY RIVER) AT LOUISA, KY.

Dam No. 3, Big Sandy River, W. Va. and Ky. View Showing the Weir Reconstructed with Steel Chanoine Wickets, (1904) (See also p. 567.)

Dam No. 1, Big Sandy River, W. Va. and Ky. View at Low Water. Total Head above Sill of Needle Pass 18 Feet (this dam Sustains at Low Water the Greatest Head of any Needle Dam yet Built); against Lock Gates, 22½ Feet. There is Practically no Leakage.

left standing and finally collected to an amount which rendered further efforts useless, and eventually wrecked much of the dam. With the 20-feet spacing of trestles adopted later there has been no trouble from drift, and each trestle can if necessary be lowered independently and in either direction.

The dam was raised $4\frac{1}{2}$ feet in 1907–1908, or to a height of $17\frac{1}{2}$ feet on the pass sill and $11\frac{1}{2}$ feet on the weir sill, making the lift 10.6 feet. The span of the trestles of the pass was changed from 4 feet to 20 feet, and wickets were substituted for needles on the weir, and the weir was provided with tripping-bars and a service bridge. The change to wickets was made to secure easier maneuvering, experience with the weirs of Dams Nos. 1 and 2, described in the following paragraphs, having shown that the reduction of leakage, which originally led to the use of needles, could be secured equally well with steel wickets. The dam, as reconstructed, embodies the same principles as Dams Nos. 1 and 2, and the system represents the most advanced type of needle dam yet built.

There are now (1912) five of these dams on the river and its two tributaries, the Levisa and Tug Forks, and a description of one is practically a description of all, except that the passes of the dams on the tributaries are only 110 feet long while that at Louisa is 130 feet long, and their weirs are also considerably shorter. Each dam (except as just noted) comprises a pass for navigation 140 feet in length and a weir varying in length from 160 feet at No. 1 to 136 feet at No. 2 and 140 feet at No. 3, a pier 10 feet wide separating the two parts. The passes are closed by needles supported at the top by girders which in turn are supported by trestles. The weirs are of the Chanoine wicket type, the wickets being of steel. The general details are as follows:

Trestles.—(See Pl. 59). The pass trestles are spaced 20 feet between centers and connection is made between their heads by means of horizontal girders, known as "escape-bars." These bars support the tops of the needles forming the dam, the bottoms resting against a sill in the masonry, and by raising the downstream side of each bar, which revolves on horizontal axes at its upstream side, the needles fall downstream, thus releasing the water from the pool above.

The wide spacing of the trestles has proved of marked advantage, not only because of the largely decreased number to be raised and lowered, but also because the danger from drift is almost entirely eliminated, as even when a piece strikes or lodges against the trestles they are sufficiently strong to withstand the shock without serious injury. The girders, or escape-bars, are hinged to the heads of the trestles by a pin held by an arm projecting upstream at each end of the girder, and resting in an open slot located on the upstream side of the trestle-head. To remove the bar and release the needles when opening the dam, power (a derrick carried on a boat) is applied to the downstream edge of the girder, which is then raised, revolving around the hinge-pins which remain in their slots until the bar

or girder reaches a vertical position, and the bay of nineteen needles has passed under it and into the river below the dam. The bar is then swung on to the boat by the derrick, where it is stored. The trestles have no axles, and are hinged to the masonry floor by means of journal-boxes and pins. They are raised and lowered by means of a winch located on the outer lock-wall and a continuous chain passing through and attached by a simple latch to the head of each trestle. The attachments may be made by these latches at any point desired when the trestles are being lowered, but it is customary to allow one trestle to almost reach the floor before connecting the next. In this way, two trestles are being maneuvered simultaneously. In raising the trestles each one is detached from the chain when it reaches an upright position, and the escape-bar is then put in place.

A wooden walk-way was originally provided on top of the escape-bars, but it has not been used, as the men in charge of the dam prefer to walk directly on the girders.

Needles.—The needles are of longleaf yellow pine, having a maximum section of 12 by 12 inches, and are provided with iron handles and rings for facilitating the maneuvers. Their weight is about 900 pounds apiece. The upper 5 feet tapers to 6 by 12 inches. The length is somewhat over 18 feet, about 5 inches of which laps on the sill at the bottom, and about 2 inches touches the escape-bar at the top. The pool rises to the level of the tops of the escape-bars, and flows over them when flush.

The needles are placed one by one by the derrick-boat by hoisting them by a ring attached near their centers and lowering them into the water, the head being steadied by a man on the escape-bar using a special lever. The first ones are placed close together, but when a head of about a foot has been obtained they have to be spaced a little distance apart so as to break the sideways current, and they are then pushed close by a long wooden lever worked horizontally from the boat. At the end of the operation the head may reach 5 or 6 feet, and the lock gates are sometimes left open to keep it down as much as possible. When all the needles are in place the lock gates are closed, being checked by lines and blocks attached near the top and bottom.

At the reconstructed Louisa dam "forked" needles have been used with much success. They consist of two long and one short needle bolted together so as to make a single needle 3 feet wide. The short middle needle leaves a space at the top 1 foot wide and 8 feet high: this is closed by a special plank placed and removed like an ordinary needle. About one-half of the pass needles are of this type, and the arrangement has been found of great value in passing small floods. These forked needles are placed and removed as just described for the ordinary needles, and with little, if any, more difficulty. Their value as regulators of pool levels is so great that it is proposed to install them at all the dams on the river.

The needles are removed, as before stated, by revolving the escape-bar till it

clears their heads, nineteen thus going out at one time. Those immediately opposite the trestles are lifted out with the derrick separately, as they do not rest against the escape-bar, and do not go out with the others. Each bay of nineteen needles is strung together on a line or chain which is attached to a long rope, securely fastened to the lock wall below the dam, and although they go out with great violence, the line draws them into the quiet water below the lock or below the part of dam remaining up, where they are held until they can be cared for.

This method of removing the needles, while very rapid and certain in times of flood, is more or less injurious to the needles themselves, as they strike against the trestles and each other with great force. Sometimes, particularly at the lower dam, where there is at this time (1912) no lower pool, they break their fastenings and go out into the Ohio River, where they are usually caught by boatmen. For this reason it is proposed to equip the service boats with power and appliances for pulling the needles upstream, one at a time, as was done at the original Louisa dam. In ordinary times of lowering this method will open the dam with sufficient rapidity, but in quick rises the escapement method will be resorted to.

Wickets.—The weir is closed by Chanoine wickets built of steel shapes and buckled plates as shown on Pl. 63*a*. They are placed 4 feet center to center and have a 3-inch space between, which is closed by a wooden joint cover during low water at Dams Nos. 1 and 2, while at Dam No. 3 and on the tributaries, bent steel plates are fastened directly to the wicket and remain permanently attached to it.* (Fig. 209*a*.) When necessary small wooden strips are used in addition in order to cut off all leakage. These arrangements make a very tight joint. The wickets have a maximum vertical height above the sill of $11\frac{1}{2}$ feet. They are raised by the derrick-boats before mentioned at Dams Nos. 1 and 2, and from service bridges at the others. A service bridge for Dam No. 1 was completed in 1912, but none is contemplated thus far for Dam No. 2.

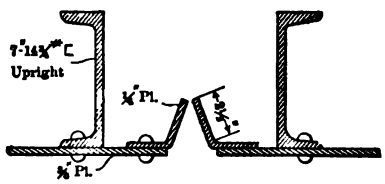


FIG. 209*a*.—Horizontal Section through Two Adjoining Wickets.

The wickets are lowered by means of tripping-bars, which are described further on.

The sill, to which the wickets are attached by pins, is of rolled steel shapes, and is continuous, and the journal-boxes are riveted to it. This gives great stability, and the wickets have no tendency to the sidewise movement so often observed in Chanoine dams, and which is the cause of the wide space being left between the wickets. Experience here has proved that no space need have been provided beyond a very slight clearance. This is of great importance for rivers having a small low-water discharge (which is the case in the Big Sandy), as there will be less opportunity for waste of water. The wickets are so designed that only the skin plate and not the main framework

* This and several other practical ideas which have assisted in and assured the success of these high needle dams was suggested by the Head Lockmaster, Mr. T. J. Snyder, of Big Sandy River.

Pass Trestles from Above, Showing Escape-bars and Footway.

Pass Trestles, Escape-bars and One Needle. Trestles are 20 Feet Centers, and the Needles are 12"×12" and a little more than 18 Feet in Length. (See Pl. 59.)

NOTE.—The footway across the pass was discarded after a short trial, as being in the way of the escape-bar maneuvers. The type of escape-bar shown was designed for use as a car track. In one of the later dams it was changed to an ordinary trussed beam with loose walk-plank. For general view see p. 568.

VIEWS OF LOCK AND DAM No. 1, BIG SANDY RIVER CATLETTSBURG, KY., COMPLETED IN 1904.

Downstream View of Wickets on Weir. (Steel Wickets.) Depth on Sill, 10½ Feet.

General View of Lock and Dam. Pass 140 feet long; weir 160 feet long. Pool partly filled.
(See also p. 568.)

VIEWS OF LOCK AND DAM No. 1, BIG SANDY RIVER, CATLETTSBURG, KY., COMPLETED IN 1904.

laps over the sill, leaving no space at the bottom between the sill and the wooden joint-covers. (See Pl. 63*a*.) This device stopped practically all escape of water and made the wickets as watertight as the needles.* The hurters are of the Pasqueau or self-tripping design. Each wicket is provided with a $\frac{5}{8}$-inch chain attached to it at sill level, the other end of the chain being held accessible in a catch on the top of the adjoining wicket, so that if a wicket is lying down, it can be raised by taking the upper end of the chain from the next wicket standing and pulling on it with the engine on the boat. Where there are service bridges the free ends of the chains are attached to the trestles in the customary way. Regulation may be done by the usual method of putting the wickets "on the swing"; experience here has shown that they will then stay lying along the props in an inclined position without any special safeguard or attachment, as the direction of the down-flowing water holds them in that position securely. They can be righted by pulling up the head or pushing down the butt. However, in streams of this nature with more or less floating drift it is unsafe to leave the wickets on the swing because even small branches or pieces which become entangled in the horses may prevent both the proper seating of the wicket when it is righted, and the proper bedding of it when it is lowered. The power for raising the wickets on the weirs having service bridges is supplied by a stationary winch on the abutment, a wire line running from it through a sheave fastened on a car which moves on the track of the service bridge.

Tripping Bar.— A steel bar having a cross-section of $1\frac{1}{2}$ by 5 inches, and divided into two sections, is used for lowering the wickets. These sections are moved by gearing placed on the pier and abutment, and operated by hand. The bars have teeth or lugs, so spaced that they will engage the props of the wickets one after another, and pull them sidewise into the downward slide, not unlike the system on the Meuse, in Belgium. The first movement throws only one wicket; but as the travel continues and the head above is reduced, the spacing of the teeth causes two or more wickets to fall at once, and the last movement throws four wickets simultaneously. The gearing was originally made of cast iron, but this was found to be too weak, and it has been replaced by cast steel in the lower portions, which are usually under water. This leaves the weakest point above water so that should breakages occur they will be accessible for repair. (See also "Tripping Bar," p. 589 and after.)

Service Bridges.—The service bridges differ from others described herein in that the spacing between trestles is increased to 16 feet and the trestles when down do not lie one upon another. (See Pl. 63*a*.) Each walkway connecting the tops of trestles is hinged to and goes down with the trestle when lowered. When the bridge is up the frame of the walkway forms a continuous track upon which a car travels which has the same wheel base as the span between the trestles. This car carries

* The senior author, Mr. Thomas, desires to credit the junior author, Mr. Watt, with originating this device and many others which have proved of permanent value in movable dams, but with which he, Mr. Thomas, has been credited by reason of having been his official superior at the time.

four sheaves (one opposite each wicket in the bay) around which a line is placed successively as each wicket is to be raised. This line is connected with the wicket chains, its free end leading back along the bridge to a stationary winch on the masonry. The car is, of course, fastened to the trestle heads while under strain. After raising four wickets the car is moved to the next span or bay and the operation is repeated. Experiments were also made with a car of similar length to the one just described, but provided with a winch operating a shaft to which chain- or pocket-wheels were keyed, one being placed opposite each wicket. By dropping the wicket chains into the wheels successively and starting the winch the wickets could be brought to position. This device is quite heavy and difficult to move, and for this reason it has not met with favor, the sheave-car being much more easily handled.

Maneuvers.—If the river is beginning to rise when the dam is in position the cover planks over the spaces in the "forked" needles are removed and the valves in the lock are opened. If the rise continues the tripping-bars are brought into play and one or more wickets are "tripped," beginning with the one next to the abutment. On those weirs having service bridges the trestles are lowered as soon as it is certain that a flood is coming, since the bridges are only useful for raising the wickets and for regulation. After the weir has been lowered the needles in the bays are connected by lines and the service-boat is placed on the upstream side of the pass and about 30 feet from the pier, with its bow towards the latter. The derrick line is then attached to the downstream side of the escape-bar next to the pier and the line is hauled up by the engine until the escape-bar revolves upstream and stands on edge. This movement gradually raises the upstream member of the escape-bar above the heads of the needles resting against it (as the hinge pins are placed some distance upstream—see Pl. 59) and the needles fall over and are carried into the eddy below that part of the dam still standing, and finally are tied up below the lock. The escape-bar is lifted clear of its bearings and swung around to its storage place on the boat, which is then moved back 20 feet nearer to the lock, and the maneuver is repeated for the next bay of needles. This leaves the trestle nearest the pier standing in the violent current caused by the release of the water, and if drift has collected it will either force its way through the opening or become lodged against the trestle. The top of the latter is connected with the pier by a chain passing over a sheave in the masonry and with heavy counterweights at its end. If drift catches against the trestle it must be dislodged, after which the trestle is lowered until its head is about 2 or 3 feet from the masonry sill. This lowering is done from a stationary winch on the lock wall by paying out a chain to which each trestle head is securely fastened by a latch just before lowering, the movement being governed by a friction brake. After the next, or third, bay of needles has been released a

slacking of the brake on the winch will allow the next trestle to start downward, since the chain is still carrying the weight of the first trestle which was not completely lowered. Thus each trestle can be lowered as soon as the adjacent bays of needles have been released.

In raising the dam the pass trestles are hoisted by the winch on the lock wall, the chain connections or latches being released as each trestle comes to the vertical, and the escape-bars being put in position by the service-boat. The needles are then placed one by one, as described in a preceding paragraph, and after they are in position the weir wickets are raised either by the service-boat or with those dams having bridges from the service-bridge.

The operation of these dams has proved satisfactory, and not as difficult as with some of the dams of less lift elsewhere. Skill and watchfulness have of course been always necessary to avoid trouble from drift and quick rises, but no more so than with the ordinary type of movable dam. By using trestles spaced 20 feet apart instead of the usual 4 feet the number required at Dam No. 1 was reduced from 34 to 6, and the weight of the ironwork was lessened by more than 25 per cent. The time taken to raise or lower the six trestles is about one-quarter of what would have been needed with a 4-ft. spacing.

Dam No. 1 during low water in the Ohio has several times sustained a difference between pools of 21 feet. (See p. 568.) Dams Nos. 1 and 2 and the locks are built on rock, as is also the dam at Louisa, and that on Levisa Fork; the lock and dam on the Tug Fork are on gravel.

Examples of needle dams are also to be found on the Ouachita River in the Mississippi basin, with trestles spaced 8-feet centers, and at a few other localities in the United States and Canada. The head of water sustained and the depths on the sills are, however, in no case as large as with the Big Sandy dams.

Time Required for Operation. See p. 556.

Needle Dams in Europe.—The most extensive system of needle dams of earlier design was to be found on the Meuse in Belgium, their construction having been completed about 1878. There were twenty-seven in the system.

The first dams on this river were closed entirely by needles, and in 1866 the system was continued by the construction of three Chanoine wicket dams. As these did not prove entirely suitable, the work was completed with nine dams using needles for the passes and wickets for the weirs. In these latest works the locks were 39 feet wide, with an available length of 328 feet, and a depth of 6.9 feet on the lower miter-sill, the upper and lower sills being on the same level. The passes were 150 feet in length, with weirs 179 feet in length, and in some cases a fixed weir was provided also. The pass trestles were spaced about 4 feet apart, and weighed about 1100 pounds apiece with all attachments, being raised by the method of separate chains and a portable winch. The needles, which

supported a head of 8.2 feet with about 10.2 feet on the sill, were 12.3 feet long and $3\frac{15}{16}$ inches wide, with a maximum thickness of $4\frac{9}{16}$ inches, and weighed about 55 pounds. They were supported by escape-bars on the Kummer system, and were removed by releasing the bar, allowing the needles to pass downstream.

The weirs were provided with wickets 4 feet 3 inches wide, and 7 feet 4 inches high, with a space of 4 inches between each, and were provided with butterfly valves in the upper parts, for the easier regulation of the pools. They were raised and maneuvered from a trestle bridge, and lowered by tripping-bars.

On the lower Seine was another system largely composed of needle dams, (see cut on p. 562) but these were replaced between 1878 and 1888 by dams of much

Dam Composed of Needles and Boulé Gates, on the River Moskva, Russia.

higher lift, closed by gates or curtains, only three dams being now (1912) closed by needles. These support lifts of 9.1 feet, 9.3 feet, 10.5 feet.

On the Moldau and Elbe in Bohemia a system has been constructed which includes several needle dams, and similar works are in operation on the Ems (see Fig. 211, p. 579), the Oder, the Main, etc.

Other examples are to be found in France, Russia, and other countries. Near Ravenna in Italy is an irrigation dam of needles supported by fixed trestles made of reinforced concrete. The trestles are spaced about 10 feet centers and are about 8 feet in height.

Limits of Lift.—The maximum lift heretofore used for needles has been that at Dam No. 1, Big Sandy River. The normal difference of the pools will eventually be 15.2 feet. (See pp. 568 and 572 and Pl. 59.) This, however, is too great a lift to allow the needles to be used for regulation, and where intended for such use

the gross head on the needles or the distance from sill to upper pool level has rarely exceeded 8 to 10 feet. In such cases the needles have been made of the hook type and square in section. The head on the needles of the regulating weirs of the Libschitz Dam in Bohemia is 10.2 feet, the lift between pools being $12\frac{3}{4}$ feet. The hook type of needle is now used on practically all needle dams recently built in Europe, owing to their great advantages for easy maneuvers.

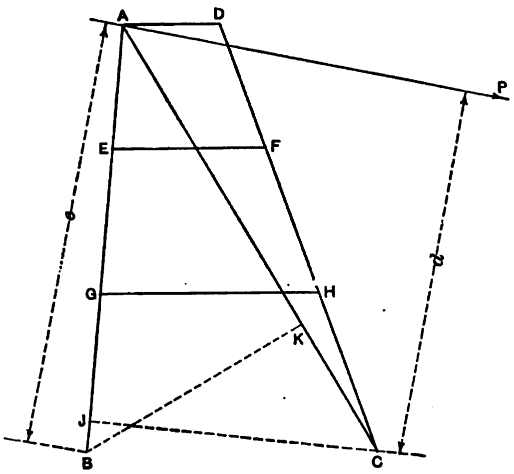


FIG. 210.

Calculations for Trestles.—The trestle of a needle dam is practically a cantilever girder or truss, supporting a load at its end. It may thus be treated as an ordinary truss with tension and compression members and stiffening braces.

If $BADC$ (Fig. 210) represent a trestle supporting a load P from the needles, AB will be in tension and AC in compression, while ADC will serve principally to support the footway or track, and the other members, as EF and GH, bind the whole together. These last must be made strong enough to support blows from the drift, etc., and are usually made of the same section as AB for trestles spaced 4 feet apart.

To find the strains in AB and AC, taking moments about C we have

$$P \times d = AB \times CJ, \quad \text{or} \quad AB = \frac{P \times d}{CJ} \text{ (tension).}$$

Similarly for AC, taking moments about B, we have

$$P \times e = AC \times BK \quad \text{or} \quad AC = \frac{P \times e}{BK} \text{ (compression).}$$

Then if l = length of AC in feet and r = the radius of gyration of its section in inches, the ultimate strength per square inch may be found from the usual formula,

$$\frac{50000}{1 + \frac{(12l)^2}{24000 \times r^2}}$$

Many needle trestles have been built with a second diagonal brace from B to D. In this case the strains will be divided between BA, AD, DC, AC, and BD, and may be found from equations of moments or by graphics. It is preferable, however, to use the simpler form of trestle, as it requires fewer pieces, and these, having in consequence to be made stronger, will resist shocks and wear better.

The omission of a continuous axle from B' to C for all trestles except perhaps for those of Boulé and Cameré dams is also very desirable, and trestles have been in successful use for many years, as on the Big Sandy River (see pp. 566 and 569), without such axles. The use of them complicates and adds to the ironwork, and in

Length between abutments (one span only used) 100 ft.
Trestles about 4 ft. (1.20 m.) centers.
SECTION OF NEEDLE DAM ON THE EMS AT VERSEN (BUILT ABOUT 1895)
Scale 1 to 300

FIG. 211.

some cases small pieces of drift have caught under them and given trouble in lowering. At the Villez dam on the Lower Seine several axles were bent from this cause.

In all ordinary cases, as in dams with trestles 4 to 8 feet apart, it will be found that a considerable excess of metal must be used above that required for the direct strains, in order to receive stiffness. Thus on the Louisa dam (Big Sandy River) the original pass trestles were 4 feet apart, and each leg was composed of one 4-inch 8-pound channel. The actual section needed for the direct strains was only about one-third of this.

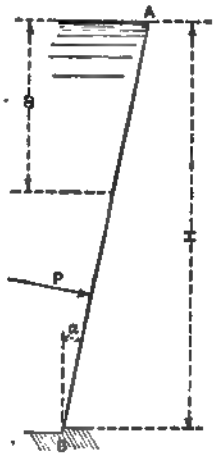


FIG. 212.

On the weir of the same dam the trestles supported a head of 7 feet, but were made of similar section, and were later used without change to carry a span of 8 feet, the original span having been 4 feet.

On the older needle dams of the Seine and of the Saône the stresses in the main members varied from 2000 pounds to 2700 pounds per square inch.

The width of base is usually made from six- to eight-tenths of the height.

Needles.—Suppose it is desired to design a needle to support a head of water H, the lower pool being left out of the calculation (Fig. 212). Let H = depth of water on the sill in feet; P = pressure of water on the needle; w = width of the needle in feet; t = thickness in inches required at the point of maximum moment. The length of the needle is then $H \sec \alpha$.

Then $P = H \sec \alpha \times w \times \frac{H}{2} \times 62\frac{1}{2}$ lbs. $= \frac{wH^2 \sec \alpha}{2} \times 62\frac{1}{2}$ lbs., of which one-third goes to A.

The bending moment M, at any point vertically distant x from the surface, is in inch-pounds.

$$M=\left(wH\cdot\sec\alpha\frac{H}{2}\times 62\tfrac{1}{2}\text{ lbs.}\times\frac{x\sec\alpha}{3}-wx\sec\alpha\cdot\frac{x}{2}\cdot 62\tfrac{1}{2}\text{ lbs.}\cdot\frac{x\sec\alpha}{3}\right)\times 12$$
$$=\frac{wx\sec^2\alpha\cdot 62\frac{1}{2}\text{ lbs.}}{6}(H^2-x^2)\times 12.$$

For a beam supporting water level with its top and one side only, this moment is a maximum at a point vertically distant $\frac{H}{\sqrt{3}}$ from the surface of the water. Putting $x=\frac{H}{\sqrt{3}}$, we find

$$M=\frac{wH^3\sec^2\alpha\times 62\frac{1}{2}\text{ lbs.}\times 12}{9\sqrt{3}}\text{ inch-pounds.}$$

From the formulas for beams we have

$$M=\frac{SI}{c}\quad\text{and}\quad I=\frac{w_1t^3}{12},$$

where S = extreme fiber stress per square inch;
c = distance of center of gravity of section from outside in inches;
I = moment of inertia of section;
w_1 = width of the needle in inches; and M and t are as before stated.

Now $c=\frac{t}{2}$, and S must be assumed as desired, say 1000 pounds per square inch, or more, according to the timber to be used (see also "Unit Stresses," p. 464). Combining the equations for I and M, we find

$$M=\frac{1000\times w_1t^3}{12}\times\frac{2}{t}=\frac{1000\times w_1t^2}{6},\quad\text{or}\quad t^2=\frac{6M}{1000\times w_1},$$

from which t can be found.

For a head of water 18 feet, with the needle slightly inclined, t was found to be $11\frac{7}{8}$ inches, w_1 being 12 inches, and S about 1000 pounds.

The usual form of needles adopted in European practice is of square section, varying from $1\frac{1}{2}$ by $1\frac{1}{2}$ inches on the first dams, with a length of $8\frac{1}{4}$ feet and a weight of 5 pounds, to $4\frac{3}{4}$ by $4\frac{3}{4}$ inches, on the more recent ones, with a length of $16\frac{1}{2}$ feet and a weight of 104 pounds. Small dimensions have been a desideratum with European designers, as the operation of the dams has been by hand, and it was necessary to limit the size of the needles to such as could be controlled by one man.

The lap on the sill is made from 4 to 6 inches.

Where there are appliances for handling them, needles of wide face are much preferable to narrow ones, as there are fewer joints and less tendency to warping, thus

GENERAL VIEW OF THE NEEDLE DAM AT KLECAN, BOHEMIA, TAKEN FROM THE LOCK WALL, SHOWING THE METHOD OF OPERATING THE TRESTLES BY A PORTABLE WINCH.

This method has since been replaced by one long chain operated from the pier. The dam consists of two weirs each 127 feet long and one pass 131 feet long. The trestles are spaced 4 feet 1 inch apart. (See Pl. 56.) The work was completed in 1899.

securing a minimum of leakage. With narrow needles, such as are generally employed, it is necessary in a dry season to use sawdust, ashes, fine gravel or similar means, to close the spaces between them. In any case, the thickness of the needle must not be greater than its width, or the pieces will turn in the current when being placed.

As regards the kind of timber, white pine appears to be the most satisfactory, as it possesses the greatest strength, weight for weight, and does not splinter easily. It is, however, somewhat costly. Oregon fir would probably make an excellent needle. Georgia pine, while possessing less tendency to become water-soaked, splinters easily, and the section required to support a given head of water will weigh more than the section which would be required in white pine. It has, however, been used very satisfactorily on the Big Sandy dams. Oak is not suitable, as it becomes water-soaked and is very liable to warp, the last disadvantage belonging to yellow poplar also, unless sawed from large trees. On the recently completed dams on the Moldau and the Elbe, in Bohemia, larch-wood was employed, the largest needles being about 4½ inches square and 13 feet long, and weighing when wet 72 pounds. On other dams in Europe Riga fir has been generally used.

The section used for needles varies somewhat; some being square, and some wider than their depth, while another form has a uniform width, but is larger at the point of greatest resistance than at either end. Hexagonal and semi-hexagonal needles, and needles with rubber upstream facings overlapping their neighbors, have been proposed and experimented with, but none of these have come into general use. A hollow needle made of four planks nailed together and banded with iron has also been proposed, but it is doubtful if it would stand handling satisfactorily.

General Remarks on Needle Dams.—The needle dam offers many advantages for rivers of moderate size and of small low-water discharge, as it can be designed to permit very little leakage, and is inexpensive in construction and operation. Among European engineers, as described on p. 542, it has come to be considered as one of the most desirable types. It is not adapted to wide rivers, however, especially where the rises are rapid, owing to the number of pieces to be handled and cared for in a short time, and it is probable that in the United States their use will be confined to the smaller streams. On the Moldau system, however, completed about 1904, Boulé gate dams were used as well as needle dams, and after several years' comparative trials it was decided that the needle type was the more economical and efficient. This river is subject to quick rises, but is not of great width. Hook needles were used in almost all cases.

The needle dam has not been developed to high lifts in Europe, since the engineers have hitherto retained the view that each needle should be controllable by one man, but the example of the Big Sandy dams has shown that this is not a desideratum, and that with the assistance of machinery the size of needles and depth of water can be increased considerably and with success.

DIMENSIONS OF TRESTLES OF NEEDLE DAMS

Location.	Height.	Width of Base.	Ratio of Base to Height.	Distance, C. to C.	Lift.	Weight Each, Pounds	Remarks.
Original Poirée trestles, about 1834 (See p. 560.)	6′ 3″ to 6′ 6″	4′ 11″	$\frac{8}{10}$	3′ 3″	3′ 3″ to 3′ 6″	220 to 310	Frames of bar iron, $1\frac{9}{16}$″ sq.
Meuse Ardennaise	8′ 0″	5′ 3″	$\frac{66}{100}$	......	5′ 10″	300	
Martot (Lower Seine)	11′ 0″	8′ 2″	$\frac{74}{100}$	......	9′ 10″ on sill	470	
Belgian Meuse	13′ 2″	8′ 4″	$\frac{68}{100}$	3′ 11′	8′ 3″	800	Footway 44″ wide, 1′ 8″ above pool.
Louisa, Ky., U. S. A. (Big Sandy River) pass, as originally built (1896). (See p. 563 and Pl. 58.)	15′ 2″	9′ 10″	$\frac{65}{100}$	4′ 0″	13′ 0″ on sill	1150 complete	Footway 38″ wide, 1′ 6″ above pool. Weights include attachments.
Do., weir, as originally built (1896)	9′ 8″	6′ 5″	$\frac{66}{100}$	8′ 0″	7′ 0″ on sill	920 complete	Footway 38″ wide, 1′ 6″ above pool. Weights include attachments.
Klecan, Moldau (Bohemia, 1900). (See p. 581 and Pl. 56.)	13′ 4″	8′ 3″	$\frac{61}{100}$	4′ $1\frac{1}{2}$″	abt. 13′ on sill	....	Footway 39″ wide, about 1′ 4″ above pool.
Dams 1 and 2, Big Sandy River, 1904, and Levisa and Tug Forks, 1911. (See p. 569 and Pl. 59.)	18′ 6″	16′ 0″	$\frac{88}{100}$	20′ 0″	18′ 0″ on sill	4000 (approx.)	Footway not generally used; worked from boat.

DIMENSIONS OF NEEDLES

Location.	Width, Inches.	Thickness. Inches.	Length.	Lift.	Weight, Pounds.	Remarks.
Original Poirée needles, about 1834. (See p. 560.)	$1\frac{1}{2}$	$1\frac{1}{2}$	8′ $2\frac{1}{2}$″	3′ 3″	5	
Meuse, passes	$3\frac{15}{16}$	Max. $4\frac{9}{16}$ Min. $3\frac{5}{8}$	12′ 4″	8′ 3″	55	Extreme fiber stress, 1060 lbs. per square inch.
Lower Seine, old dams. (See p. 562.)	$3\frac{1}{8}$	$3\frac{1}{8}$	13′ 2″	7′ 0″	......	Extreme fiber stress 2300 lbs. per square inch. Found to be too great.
Joinville, Marne, 1885	$7\frac{7}{8}$	$4\frac{3}{8}$	13′ 9″	9′ 10″ on sill	108	Replaced later by needles $4\frac{3}{4}$″ sq.
Marne, 1897	4 to $4\frac{3}{4}$	4 to $4\frac{3}{4}$	16′ 5″	......	104	Hook-needles.
Big Sandy River (Louisa, Ky., 1896), original needles of pass. (See p. 563 and Pl. 58.)	12	Bott., $8\frac{1}{2}$ Top, $4\frac{1}{2}$	13′ 10″	13′ 0″ on sill	263	Extreme fiber stress, 1230 lbs. per square inch.
Do., original needles of weir.	12	Bott., $3\frac{1}{2}$ Top, $2\frac{1}{2}$	7′ 8″	7′ 0″ on sill	80	Extreme fiber stress, 990 lbs. per square inch.
Klecan, Moldau (Bohemia, 1900). (See p. 581.)	$3\frac{3}{4}$ to $4\frac{3}{4}$	$3\frac{3}{4}$ to $4\frac{3}{4}$	10′ 9″ to 13′ 0″	10′ 0″	46 to 72	
Dams 1 and 2, Big Sandy River, 1904, and Levisa and Tug Forks, 1911. (See p. 569 and Pl. 59.)	12	12	18′ 6″	18′ 0″ on sill	900	Extreme fiber stress about 1000 lbs. per square inch.
Herkimer, N. Y., 1912. (See bottom of p. 561.)	$5\frac{3}{4}$	$5\frac{3}{4}$	12′ 9″	9′ 0″ on sill	......	Hook-needles.

COST OF NEEDLE DAMS

Location.	Lift.	Cost per Foot Run.			Remarks.
		Fixed Parts.	Movable Parts.	Total.	
Meuse Ardennaise	5′ 11″	....	....	$97.50	Average of whole dam.
Belgian Meuse	8′ 3″	$110.00	$43.00	153.00	Cost of passes.
Saône	7′ 6″	....	....	110.00	Cost of weirs.
Martot, Lower Seine	9′ 10″	....	....	244.00	Cost of pass.
Louisa, Ky., Big Sandy River (original construction; see p. 563)	13′ 0″ on sill	226.50	19.20	245.70	Average of pass and weir.
Dam No. 1, Big Sandy River. (See p. 569.)	18′ 0″ on sill of pass	....	....	347.00	Needle pass, wicket weir. Cost is average of pass and weir, and includes abutment, center pier, protection cribs, etc. Engineering and superintendence not included.

CHAPTER VII.

CHANOINE WICKET DAMS.

History.—As the Poirée dam was developed from the old-time plank dams, so the Chanoine wicket dam is the evolution of another type. Gates with horizontal axes fixed in the abutments had long been in use, as, for example, the tide gates used in the dikes of Holland, while the weirs of certain dams on the Orb (1778) were formed of shutters supported by hinges fastened to the floor. Later on M. Thénard added the prop, the hurter, and the tripping-bar, and the invention of M. Chanoine in 1852 raised the axis of rotation to near the middle of the wicket, creating a type which has remained almost unchanged.

Description.—The Chanoine wicket is a shutter hinged near its middle, supported when up by a prop, and kept in position when revolving by a hinged frame known as the horse. The lower end of the prop rests against a casting on the floor called the hurter. When this end is moved from the support, the pressure of the water pushes down the wicket, which turns on the horse until it lies flat behind the sill, with the hurter, prop, and horse underneath. The part of the wicket above the axis of rotation is called the head, or chase, and the part below, the breech or butt. (See Pls. 60 to 64 and pp. 588 and 598.)

In the heads of the wickets are sometimes placed other small wickets, known as flutter- or butterfly-valves, turning on an axis framed into the main wicket and kept closed by latches. Their object is to afford a means of regulating the pool without having to operate the large wickets, and they are opened or closed by means of a hook-pole with which the dam-tender frees or closes the latches. They have not been used in America, as drift would prove troublesome, but they are in use on the Meuse, at La Mulatière dam, and elsewhere. (See Pl. 60.)

The original idea of Chanoine was to place the axis at a point above the sill one-third of the length of the wicket, this being the center of pressure of the water. As the pool rose, this center would be changed to above the axis, and the wicket would swing automatically, reducing the pool to its proper level, when the wicket would swing back. All the early dams were constructed on this plan, being raised from boats and lowered by tripping-bars, but it was soon found that the wickets were too sensitive, and while opening without trouble, they would not swing back without assistance, or until the pool had fallen considerably. Moreover, when one wicket opened it caused the lower pool to rise, creating a greater depth of water below

the dam and thus unbalancing the center of pressure on the downstream side of the other wickets. This caused others to open, until the whole dam was on the swing and the pool levels were disorganized. For these reasons the axis was raised in all the later dams, as mentioned further on. Fixed, and in some cases sliding, weights or counterpoises were attached to the wickets, to regulate better the tendency to swing, but they did not prove satisfactory. With a view to minimizing the evil, experiments were tried on the upper Seine dams about 1865, in which several of the wickets on the weirs were provided with chains fastening the butts to the sills, and were made wider at the same time in the heads so that they would swing before the others. The chains were of such a length that when the pool rose and swung the wickets, the latter could not pass an angle of forty-five degrees with the vertical, and would therefore right themselves much sooner than when swinging free. This method was in part successful, but it was soon found to give rise to another objection, which was that the wickets by being chained reduced the area of spillway of the dam. In order, therefore, to pass rises which formerly could be disposed of by putting the wickets on a full swing, it became necessary to lower several entirely or else to maneuver the heavy wickets of the pass. The operation of raising them again, which had to be done from a boat exposed to strong currents, was found to be difficult and dangerous. Finally, as the only solution of the problem, De Lagréné proposed placing a bridge of trestles above the dam from which the wickets could be maneuvered, and such a bridge has now come to be regarded as almost indispensable for convenient operation, provided the conditions of drift, etc., are such as to permit its use. (See Pl. 61 and p. 598.)

On the Ohio River, where the passes and most of the weirs are of wickets, service-boats are used for all the maneuvers of the passes, but the service-bridge is still adhered to for the weirs. (See p. 592.)

Dimensions and Materials.—A width of 3 to 4 feet has been usually adopted for wickets, the latter width having been used on all American dams. From this is deducted a space of 3 to 4 inches for clearance between each one, this being necessary, with wooden wickets, to allow for warping and side movements during maneuvers. The width of the pass and of the weir wickets is generally the same, although in a few cases the former is less, owing to the greater height. On the upper Seine, for example, certain of the passes have wickets 11.8 feet high and 3.3 feet wide, while those of the weir are 6.5 feet high and 4.3 feet wide. The angle of inclination to the vertical, where Pasqueau hurters are used, is made about twenty degrees; but on the earlier dams, where the tripping-bar was relied on for lowering, the angle was much less. The lap on the sill is 4 to 5 inches. The largest wickets are to be found on some of the Ohio River dams and measure 3 feet 9 inches in width by about 18 feet in height, being built of timbers with a thickness of 12 inches at bottom (see Pl. 63*a*). They have been in use for some years

GENERAL VIEW OF THE NAVIGATION PASS (600 FEET LONG), DAM No. 6, OHIO RIVER, 1898, SHOWING THE CHANOINE WICKETS RAISED IN POSITION.

(Work just completed and yet in coffer-dam. The wickets were raised later about 4 feet, in order to provide a channel depth of 9 feet, making them of a total length of 17′ 10″.)

VIEW SHOWING CHANOINE WICKETS, PROPS, ETC., OF THE NAVIGATION PASS, DAM No. 6, OHIO RIVER, 1898.

(From sill to upper pool is 13 feet 2 inches.)

and in spite of their great size are handled by the service-boats without special difficulty.

The great majority of Chanoine wickets have been built of wood, on account of its cheapness and lesser weight, especially for small wickets. Those on the Ohio, Kanawha, Congaree, Brazos and Osage Rivers are of wood. On the Big Sandy and Trinity Rivers they are of steel. A description of the Big Sandy wickets has been given on p. 571 and the design is shown on Pl. 63*a*. The first metal wickets were used at La Mulatière dam near Lyons, 1879 (see Pl. 60).

Hurters (see Pl. 62).—The hurter is a casting consisting essentially of a shoulder and of two grooves, one of which guides the end of the prop when the wicket is being raised till it falls against the shoulder, while the other guides it back when released and leads it into position ready again for raising. Until 1879 all lowering had to be done by a tripping-bar, or by pulling the props sideways till they cleared the shoulder. This was facilitated by sloping the latter slightly in the horizontal plane, so the prop would bear against a slanting surface. The safe maximum of this angle, as found by experiments made many years ago at the dam of Conflans in France, was three degrees. In the year mentioned, however, at La Mulatière dam at Lyons, M. Pasqueau introduced the double-stepped hurter, which has practically superseded the former kind, and which provides a second shoulder upstream of the first one. In this case the wicket is pulled upstream until the end of the prop leaves its support and falls over the second shoulder into a groove. The wicket is then released and the prop slides along the groove which guides it back into position ready for raising, in the same manner as with the first type of hurter.

Tripping Bar.—One of the most valuable devices used in connection with the operation of a Chanoine dam, especially on the smaller rivers, is the tripping-bar. This is a bar provided with teeth and moving along just under the feet of the props. It is worked by gearing in the pier and the abutment masonry, the teeth being arranged so as to strike each prop in turn and pull it sideways from the shoulder, thus allowing the wicket to fall. A cushion of water is always desirable as the falling parts may otherwise be damaged or broken by the shock. The machinery must of course be arranged to act in either direction, since it has to push the bar back to place after the wickets have been thrown. To guard against danger from drift, etc., the bar should be made of flat section, say 5 inches wide and not less than an inch thick, the teeth being riveted on as bent plates, and it should be recessed so that its top will not project above the masonry. It may slide on rollers or on surfaces provided on the hurters, the latter plan being preferable, as the rollers are very liable to become clogged or broken. Guards must be provided every few feet to prevent the bar from lifting up; these can be bolted to the hurters.

Where it is desired to use a Pasqueau hurter, which would always be advisable

since the bar may at some time become unworkable, the latter may be arranged to slide between the two shoulders, the prop passing over it when the wicket is maneuvered, as was done on the Big Sandy dams just referred to. (See Pl. 59 and p. 571.)

As the total travel of the bar is limited to the distance between any two props minus the width of the tooth, this arrangement, as heretofore applied, is inapplicable to wide passes, since too many props would have to be thrown at once in order to get all of them down within the limit of the travel. About 200 feet is the widest opening to which it has been applied, and it has then to be divided into two parts, one working each way. At first only one tooth engages, but at the last several teeth have to be pulling together. At Mélun on the Upper Seine the bar was used on a pass 214 feet long, one section dropping 25 wickets in a travel of 38 inches. For long dams it is possible that two or more bars, placed side by side, could be operated, the props of the wickets being made of varying lengths to correspond, and suitable machinery being installed in the masonry. It would be possible also to design mechanism of sufficient power to throw a larger number of wickets simultaneously.

The tripping-bar has been in successful use on the Meuse, the Upper Seine, and elsewhere for many years. Some trouble was experienced at first with its use, owing to insufficient power and to the obstruction of gravel, but in later designs these objections were overcome, the machinery being given a force of 12 to 15 tons, and pulling the teeth through any ordinary obstacle. It has not been generally applied in America, partly owing to the width of the rivers in which Chanoine dams have been built, and partly to the fact that on the Kanawha River where it was first used the bar was too weak and was also set above the floor, as in the French designs, so that drift caught under it frequently and unseated it. Its use on this river was therefore abandoned, but the few times in which it was maneuvered showed it to be a valuable device, as the wickets could all be lowered safely from the shore and even with ice against them.

On the five Chanoine weirs of the needle dams on the Big Sandy River (built 1904–1911; see p. 569 and after and Pl. 59) tripping-bars were installed and, with the exception of the breaking in one instance of some of the cast-iron gearing (which has since been replaced with steel castings) they have given excellent satisfaction. It is necessary occasionally to clean or wash out the machinery recesses, especially at the first raising of the dam in the Spring, as the rollers behind the rack-bar, whose function is to hold the rack in contact with the pinion, move the sand forward between the rollers and the bar, and the mechanism in consequence becomes wedged so tightly that further motion is impossible. On one of the weirs the rollers were replaced later by a fixed angle, after which there was no further clogging. Angle guards are attached to the hurters in order to hold the bar in place on the

masonry apron and have been found to be quite efficient. The rack at the end of the bar and the pinion which moves it are now made of annealed cast steel, while the gearing above water which turns the shaft of the pinion is of cast iron, thus providing the weakest part at an accessible point. It is of the highest importance that ample strength be given the gearing, bearings, etc., since a breakage under strain may stop the lowering of the dam and lead to serious results.

Maneuvers.—Wicket dams of the Chanoine type are maneuvered either from a service-bridge of trestles or from a boat, the lowering in some cases being done by a tripping-bar, as described further on. Experience with the comparative advantages of operating from a bridge or from a boat appears to be conflicting, and largely dependent on local conditions. Thus on the Meuse, in Belgium, a boat is used to raise the passes of the wicket dams, the lowering being done by tripping-bars, but service-bridges were provided for all the weirs, where they were deemed necessary in order to control the wickets for the regulation of the pool. On the upper Seine, after experiments in using a boat for the weirs, it was deemed preferable to employ service-bridges, a boat being found unsuited for night-work and for easy regulation, and they were accordingly added.

A bridge was added to the weir of Dam No. 1, Big Sandy River, after a trial of several years without it, but none has been added to Dam No. 2 a few miles above. The Kanawha dams are operated altogether from service-bridges.

Where steam is employed there seems to be no special objection to using a boat instead of a bridge for operating a pass. With a weir, however, a bridge appears to be desirable (although not always necessary) as it allows an easier control of the wickets when they have to be swung to regulate the pool.

On the dams of the Ohio River the passes, which have a maximum length of 900 feet, are operated from a boat carrying suitable machinery, and usually rigged as a derrick-boat with a stiff-leg derrick and a double-drum hoisting engine and swinging engines, and spuds. The Ohio River boats are about 70 feet long, from 22 to 26 feet in width and from 3 to 4 feet in depth. When lowering the dam the boat is placed lengthwise against the upstream side of the wickets (being prevented from bearing against their upper parts by spuds which reach down below the axes) and at the end of the pass furthest from the lock (see cuts on p. 592). A rope leads from a capstan on the stern of the boat to the upper end of the lock wall, and as each wicket is lowered the rope is wound in sufficiently to move the boat backwards into proper position to make attachment to the next wicket. The lowering is done by attaching a grab-hook and line to the head of each wicket and then pulling the head upstream until the prop drops into the downward slide of the hurter. By slacking-off on the engine the pressure of the water then pushes down and beds the wicket. At the close of the operations the boat has of course reached the lock wall and is then removed to a place of safety.

Maneuvering Boat at Dam No. 2, Ohio River, raising a fallen wicket.
(The dam has just been raised and the pool is filling.)

Raising the Pass Wickets at Dam No. 2, Ohio River.

No chains are used, but the bottom of the wicket is caught by hand by a hook-pole as shown, attached to a wire line running to the engine. The boat rests against the wickets and is held also by a 2-inch manila line running to the lock-wall. (See pp. 591 and 593.) Occasionally the boat has been carried through the opening, but no damage has resulted. The hull shown is 70 feet long by 22 feet wide.

The raising of these dams is begun at the end of the pass next to the lock, the wickets being grappled or fished for by a hooked pole which must catch in the iron staple at the bottom of the wicket. A line from the hook passes to the engine through a sheave attached to a beam projecting from the bow of the boat, and the wicket is then hoisted until the prop drops into its seat in the hurter, and the hooked pole is disengaged. The wicket, which comes up "on the swing," is righted or brought to its proper position against the sill by pushing down the butt. No special precautions are taken to lessen the shock when it strikes the sill.

Much skill is required to engage the hook in the staple at the bottom of a wicket, as the water is deep and the current very swift, but the operators become very expert, and seldom make a miscalculation. There is also a certain amount of danger in having the boat moored at the edge of the opening, where there is always a strong side current. Accidents are, however, very rare, although on two occasions a boat has lost its moorings and been carried into the pool below, fortunately without any harm resulting.

Where a service bridge is employed, as is the case with nearly all small rivers, it consists of movable trestles, provided with floors or "aprons" and a service track on which the operating winch rolls. (See Pls. 61, 63, 63*a*, and pp. 597 and 598.) The trestles are placed 8 to 9 feet apart, and have their footway 2 to 3 feet above the pool. To raise the dam, the trestles are first pulled up by the winch, using the short chains attaching each trestle-head to the end of the apron of the next one, and setting the aprons and track rails in position as the maneuvers progress. The wickets are next pulled up by the breech-chains which connect the bottom of each wicket with the head of the nearest trestle, each wicket being moved forward until its prop is heard to drop over the shoulder of the hurter. When the prop is seated the wicket is left "on the swing," being held in that position by the current. In this way the pass and weir wickets can all be raised and held without obstructing the flow of the river to any appreciable extent. When all is ready, the wickets are righted and the current catches the butts and forces them against the sill, thus closing the dam. This righting is done in the same manner as when using a service-boat, namely, by pushing the butts down with a pole, which has usually to be operated by the winch, if the wickets are of any size. Another method which has been experimented with consists in one or two men walking across the wickets and overbalancing each until it strikes the current, when they jump on to the next one and repeat the maneuver until all are righted. This is, however, attended with much danger, as if the men fall into the river they will almost certainly be caught in the horses and be drowned.

To regulate the pool, a few wickets are lowered with or without lowering the trestles according to circumstances, or the wickets are merely put on the swing and kept in that position by holding the breech-chains in the "stops" or chain sockets

on the trestle-heads. When the discharge becomes low the spaces between the wickets are closed by means of square timbers, known as "joint covers," pushed down into them cornerwise.

To lower the dam, the props are displaced by the tripping-bar, or, if Pasqueau hurters are employed, the wicket is pulled upstream by the winch or from a boat until the prop falls over the shoulder into the groove, and the wicket is then lowered. On the weir this is done by pulling on the breech-chains, but on the pass, when the lowering of the weir has reduced the pressure, the head of the wicket is pulled upstream by a special grab-hook, as before described for the Ohio River dams, until the prop has fallen into the return groove of the hurter, when the hook is jerked off and the water pushes the wicket down. This saves having first to put the pass wickets on the swing, and permits of very rapid maneuvers. The breech-chains are then fastened in the stops by pins and the trestles are lowered.

In European dams entirely dependent upon the tripping-bar under ordinary conditions, if the bar gets out of order the head of each wicket is pulled upstream from a boat until its prop is clear of its support, and the prop is then pushed to one side with a boat-hook from a skiff below and the wicket is lowered.

In maneuvering this type of dam in order to pass a small rise it is frequently necessary to lower from one to ten wickets, usually next to the pier, and it is also necessary to be able to replace them one by one under practically a full head when sufficient water has been allowed to pass. Under these conditions it has been found difficult to seat the props of the wickets in the hurters, owing to the sideways currents of the water flowing through the restricted opening. To remedy this the props are usually constructed with much extra weight at the lower ends (see p. 588), but this has not always proved satisfactory. The difficulty could probably be reduced by adding the increased weight as a round rather than as a flat section, as is usually done. The most violent side currents occur adjacent to the masonry, and it has been suggested accordingly that if the pool was regulated by lowering the wickets at some distance out the trouble might be diminised.

One objectionable feature of a service-bridge is the floor sections, or "aprons," fastened to the heads of the trestles (see p. 598 and Pl. 63). When the dam is being lowered, as well as when it is down, the current almost invariably catches some of them, so that their free ends rise and swing in the water and are then very liable to be injured by drift or by boats. This trouble is noticeable on any movable dam which has trestles provided with aprons, and it occurs chiefly on the weirs, since there the currents are more violent than in the passes. Many expedients have been tried to overcome it, but none has succeeded entirely. At La Mulatière dam the rails were braced together and hinged to the trestles, and a light floor of plank, of a total width of 4 feet, was fastened between them. By this means the surface exposed to the current was reduced and the weight considerably increased. This plan appears

to have several advantages over the usual construction, among which is the fact that it avoids the handling and transportation of the rails.

La Mulatière Dam.—This dam, completed on the Saône in 1879, at the junction of the Saône and Rhone rivers just below Lyons, marked a notable advance in the design of Chanoine wicket dams. It was constructed under the direction of M. Pasqueau, and it was there that he first introduced the double-stepped hurter, which has since been almost universally adopted.

The pass of the dam is 340 feet in length, and is closed by wickets made of iron, 4.6 feet wide and 14.25 feet long, each being provided with a flutter-valve about 2.9 feet wide and 5.1 feet long, operated by a hook pole in the usual manner. The sheathing plates are $\frac{3}{16}$ of an inch thick. The wickets are operated by a steam-engine which moves on a service-bridge of trestles, the track being $6\frac{1}{2}$ feet above pool level. These trestles are 9.8 feet apart and 22.3 feet high, and were the first in which no long bottom axle was used, pins being employed instead. By this arrangement the depth of the recess behind the sill was reduced from 4 feet to 28 inches. (See Pls. 60, 62, 63 and 64.) It was here also that steam power was first employed instead of the old methods of hand power, and the success of the work generally had a considerable influence on wicket dams built since its completion.

American Dams. (See Pl. 46 for locations of streams.)—At present only one river in America, the Kanawha River in West Virginia, has a completed system of wicket dams. They are eight in number, with lifts of from $6\frac{1}{4}$ to $8\frac{1}{2}$ feet, the weirs being from 210 to 364 feet in width, and the passes from 248 to 304 feet. The system was built between 1880 and 1898, and in connection with two fixed dams affords a depth of 6 feet over about 90 miles of river. These dams are generally similar in design to those of Europe, and are operated, both for the passes and the weirs, with trestle bridges and winches as before described. The locks are 55 feet wide and 342 feet between hollow quoins. (See Pls. 60 to 67 and pp. 401 and 597–8.)

On the Ohio River a system of locks and wicket dams has been commenced.* The first one to be completed, that of Davis Island, just below Pittsburg, was finished in 1885. Between that year and 1912 ten more were finished and in the last-named year ten additional ones were under construction. The system when completed will extend from Pittsburg to Cairo, a distance of over 950 miles, and will comprise 54 locks and movable dams, overcoming a total fall of about 423 feet. The locks are all 110 feet wide with an available length of 600 feet and with lifts ranging from 5 to 11 feet, the majority being about 7 to 8 feet. The gates are of

* See Tables of Locks and Dams at the end of the book. For details of the work see "The Improvement of the Ohio River," Transactions Am. Soc. C. E., 1908, by Major W. L. Sibert, Corps of Engineers, U. S. A., by whose courtesy some of the accompanying plates are reproduced; also "Professional Memoirs," Corps of Engineers, U. S. A., October-December, 1911, article by J. W. Arras.

the rolling type and the valves of the butterfly type. There are 32 of these to each lock, 4½ feet in diameter. (See p. 492–3.) The depth on the lower sills varies from 9 to 11 feet, the intention being to provide a minimum navigable depth of 9 feet. The dams constructed or designed up to 1912 average about 1100 feet in length, varying from 892 to 5046 feet, with navigation passes of lengths varying from 559 to 900 feet (the average being about 600 to 700 feet), closed by Chanoine wickets. With the upper pool full the depth on pass sill is generally about 15.4 feet, but at some of the dams it is only 13½ feet. The wickets are from 9 feet 9 inches to 18 feet long. The weirs are divided generally into three parts, one part being closed by Chanoine wickets and varying in length from 72 to 400 feet (the average being about 200 feet), and two parts being closed by bear-traps of an average net length of 91 feet each. (Fig. 212a.) At one of the dams there is a bear-trap 52 feet in length; at another there are two each 50 feet in length; and at a third there are two each of

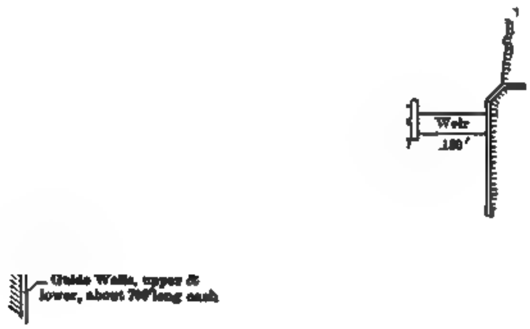


Fig. 212a.

120 feet, but the usual length is 91 feet. The bear-traps have from 9.2 feet to 15 feet of vertical depth on the sills. (See illustration of Dam No. 6 on p. 644.)

The passes are maneuvered from service-boats; the wickets of the weirs are maneuvered from service-bridges; and the bear-traps are controlled by valves located in the piers, with compressed air used as an auxiliary when required. (See Pls. 54, 62, 64, 66 and 67.)

These dams have been utilized to supply artificial floods or "splashes" as described on p. 554. Thus on one occasion a large steamboat, drawing about 2½ feet, wished to proceed down the river in order to permit an inspection of the stream to be made by the Congressional Committee of Rivers and Harbors. It was at a time of very low water, and in order to create an artificial flood sufficient to permit the boat to pass the unimproved portions of the river (about 900 miles in length), part of the water was let out of the pools of Dams Nos. 3, 4, 5, 6, and 8 by lowering a bear-trap in each dam simultaneously. The steamboat was waiting at Wheeling, 44 miles below Dam No. 8, which was the lowermost structure in commission in that portion of the river. The extreme front of the flood traveled at about 3 miles per hour, and

GENERAL VIEW OF CHANOINE DAM No. 11, KANAWHA RIVER, W. VA., U. S. A.

The view is taken from the abutment side, and the pass is seen next the lock, with the weir in the foreground. The water is about 15 inches above the pool level. The weir is 364 feet in length and the pass 304 feet. The chamber of the lock has a width of 55 feet and an available length of 313 feet. The lock and dam were opened to navigation in 1898. The lift is 10.9 feet.

VIEW OF THE ABUTMENT AND PART OF THE WEIR OF A CHANOINE WICKET DAM (KANAWHA RIVER, W. VA., 1897).

Three wickets are standing in position next the abutment; one is on the swing, held by the chain; the others are lowered behind the sill. The wickets rise 8 feet 6 inches vertically above the sill and are 3 feet 9 inches wide, the trestles are 12 feet high and 8 feet apart. The chains for maneuvering the wickets and trestles are not all in place.

the maximum stage was reached at Wheeling 29 hours after releasing the bear-traps, and amounted to a rise of 5 feet. By the aid of this flood, reinforced further down the river by water from the Kanawha River dams and from three other Ohio River dams which were in commission, the steamboat was able to complete her journey of about 900 miles to the mouth of the river.*

Design of Trestles. (See Pls. 61, 63, and 63*a*.)—The trestles for wicket dams differ from those required for other types in that the only direct load they sustain comes from the pull of raising or lowering the wickets, and from the weight of the winch and its car. The indirect loads from drift, etc., are of course the same as elsewhere.

The direct load is a variable quantity, and can only be approximated. Its maximum occurs when the wickets have to be pulled out of deposit or drift which

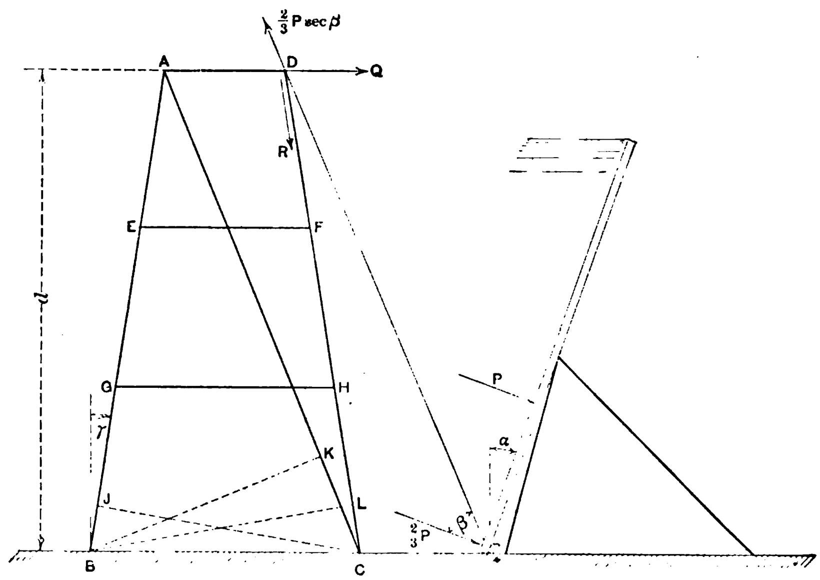


FIG. 213.

may have settled on them, and when the deposit is of any depth the winch is sometimes unable to move them. The load should therefore be assumed as not less than the maximum capacity of the winch, and it will usually be found sufficient to take it as equal to two-thirds of the total water pressure on the wicket when standing, multiplied by the angle between the chain and this pressure. The angle may be assumed to be the same when the wicket is upright as when it is lowered, the actual difference being small.

Thus in Fig. 213, according to this supposition, the direct load on the chain will be $\frac{2}{3}P \cdot \sec\beta$, where P = the total water pressure on the wicket, and β = the angle between the chain and the direction of P. The water should be assumed at pool level, or above, with no water below, so as to provide for the most unfavor-

* Annual Report, Chief of Engineers, U. S. Army, 1912, p. 2327.

able condition. This load is usually a maximum on the weir of a dam, as its wickets are lowered first, and by the time the pass is reached the difference between the upper and lower pools has been much reduced. The winch, however, should have considerably more than the theoretical power required, and the trestle should be designed accordingly, since, as before mentioned, the wickets have sometimes to be pulled out of deposits of sand or mud.

To find the strains in the trestle, assume the line of pull to pass through D and resolve it there into two components, Q and R, along AD and DC. R is then taken up directly by the member DC.

Taking moments about C, we have

$$Q \times d = AB \times CJ, \quad \text{or} \quad AB = \frac{Q \times d}{CJ} \text{ (tension).}$$

Taking moments about B, we have

$$Q \times d = AC \times BK, \quad \text{or} \quad AC = \frac{Q \times d}{BK} \text{ (compression).}$$

The vertical upward pull on the anchorage at B where γ = angle of inclination of AB = strain in $AB \times \cos \gamma$.

As mentioned in the calculations for needle trestles, it will be found that a considerable excess of strength must be supplied for stiffness, and experience can prove the only guide in such matters. On the Meuse dams, with a head of about $7\frac{1}{2}$ feet, the trestles were made in the form of a St. Andrew's cross, of bar iron about $1\frac{1}{2}$ inches by 2 inches. On the Saône, on the pass of the Ile Barbe dam, the trestles were about 13 feet high, and made of channel-irons about $2\frac{1}{2}$ inches wide. On the Kanawha River the pass and weir trestles are 16 feet 9 inches and 11 feet 9 inches high respectively. They are built in the shape shown in Fig. 213, DC on the pass being composed of two 4-inch 9-pound channels and all the other members of one channel of similar size. On the weir, DC is formed of a 3-inch 9-pound I beam, and the other members are of a single 3-inch 6-pound channel. The trestles are all 8 feet apart. (See Pl. 63 and also Pl. 63*a*.)

DIMENSIONS OF TRESTLES OF CHANOINE WICKET DAMS

Location.	Height.	Width of Base.	Ratio of Base to Height.	Distance between Centers of Trestles.	Lift.	Weight, Each, Pounds.	Remarks.
Belgian Meuse............	8′ 2″	5′ 5″	$\frac{66}{100}$	4′ 8″	7′ 6″ on sill	......	Footway 44″ wide, 1′ 8″ above pool.
Port-à-l'Anglais, Paris, 1870..	15′ 9″	10′ 2″	$\frac{64}{100}$	3′ 7″	9′ 10′	1325	Footway 33″ wide, 1′ 8″ above pool.
La Mulatière, Lyons, 1879...	22′ 0″	11′ 6″	$\frac{52}{100}$	9′ 10″	11′ 10″ (max.)	......	Footway 48″ wide, 6′ 6″ above pool.
Kanawha River, W. Va., Pass of No. 7, 1893.......	16′ 10″	11′ 10″	$\frac{65}{100}$	8′ 0″	13′ 0″ on sill	about 1800	Footway 29″ wide, 2′ 6″ above pool.

The trestles of La Mulatière dam, near Lyons, are 9.8 feet apart, in the form of a St. Andrew's cross, and carry a steam engine with which the dam is operated. Each member consists of a single channel-iron, $5\frac{1}{2}$ inches wide. Their height is a little over 22 feet, and the width of base 11 feet 6 inches. (See Pl. 63.)

Design of Wicket.—*General Strains.* (See Pls. 60 and 63*a*.)—To find the maximum strains in a wicket of the Chanoine type we will assume the water to be flowing over the top to a depth d, and that there is no water below. In practice d may vary from 0 to $1\frac{1}{2}$ feet (see Fig. 214 and p. 597).

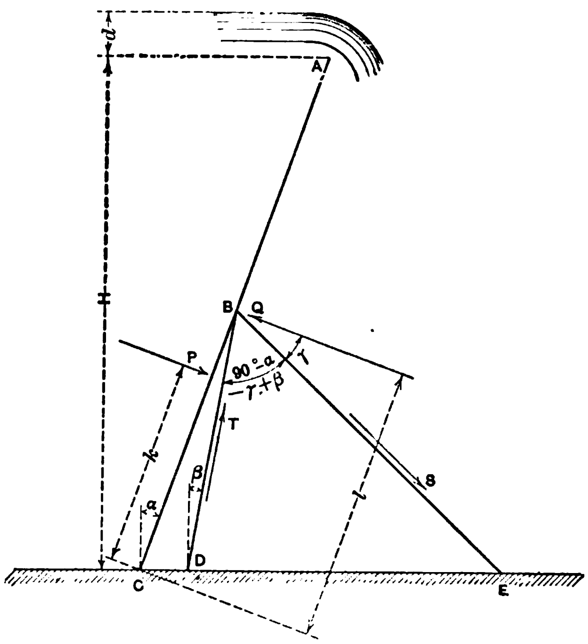


FIG. 214.

Let H = depth of water on sill in feet;
d = depth of overflow in feet;
w = width of wicket in feet;
P = total of pressure of water;
Q = that part of P acting at B;
S = strain in prop;
T = strain in horse;
α, β, γ = angles of inclination as shown,
k = distance from base to center of pressure;
l = distance from base to center of support.

Then

$$P = w \times H \sec \alpha \times 62\tfrac{1}{2} \text{ lbs.} \times \left(\frac{H}{2} + d\right).$$

The distance k may be found by considering P as composed of a rectangle and a triangle of pressures (Fig

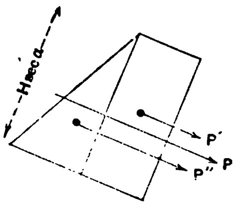


FIG. 215.

$$P=P'+P''=dwH \sec \alpha \times 62\tfrac{1}{2} \text{ lbs.} +wH \sec \alpha \cdot \frac{H}{2} \times 62\tfrac{1}{2} \text{ lbs}$$

Then, taking moments about C, the moment of P = sum of moments of $P'+P''$, or

$$P \cdot k=P' \times \frac{H \sec \alpha}{2}+P'' \times \frac{H}{3} \sec \alpha=\frac{wH^2 \sec^2 \alpha}{2}\left(d+\frac{H}{3}\right) \times 62\tfrac{1}{2} \text{ lbs.}$$

Whence $k=\dfrac{wH^2 \sec^2 \alpha}{2P}\left(d+\dfrac{H}{3}\right) \times 62\tfrac{1}{2} \text{ lbs.}=\dfrac{H(H+3d) \sec \alpha}{3(H+2d)}.$

To find Q, take moments about C, whence

$$Q \cdot l=P \cdot k, \quad \text{or} \quad Q=\frac{wH^2 \sec^2 \alpha}{2l}\left(d+\frac{H}{3}\right) \times 62\tfrac{1}{2} \text{ lbs}$$

Then from the proportion between the sides and sines of a triangle,

$$S=\frac{Q \times \sin\left(90^\circ+\alpha-\beta\right)}{\sin\left(90^\circ-\alpha-\gamma+\beta\right)}=\frac{Q \cdot \cos(\alpha-\beta)}{\sin\left(90^\circ-\alpha-\gamma+\beta\right)} \text{ and } T=\frac{Q \cdot \sin \gamma}{\sin\left(90^\circ-\alpha-\gamma+\beta\right)}.$$

The pressure of the wicket against the sill at $C=P-Q$, and the upward pull on the anchorage at $D=T \cos \beta$.

In the preceding calculations we have neglected any water pressure below the wicket, in order to obtain the maximum strains which can occur. With the axis of rotation placed at the height dictated by experience and mentioned a little farther on, it has been found that wickets do not trip themselves under any ordinary conditions of practice, and that they will support without derangement a pool-level 12 to 18 inches above their tops. The latter condition is one which sometimes proves of much benefit, not only when an unexpected rise comes, but also where an extra channel depth is required for a short while to float too heavily loaded craft.

At the mouth of a stream, however, tributary to a large river which may cause backwater during a flood, the tributary itself being at an ordinary stage, it will be necessary to adjust the height of the prop so that the wicket will not trip before the backwater is level with the pool above. Unless this is provided against the wickets will begin to swing themselves too soon, thus reducing the upper pool level and the depth of the water on the shoals and at the lock above. Such cases have occurred and have caused trouble to craft.

To find the height at which the prop should be placed in order to avoid this self-operation, let P, H, d, etc., represent the same quantities as before, and let h represent the depth of the water below, and R its pressure (Fig. 216).

Then $R = wh \sec \alpha \times 62\frac{1}{2} \text{ lbs.} \times \frac{h}{2} = \frac{wh^2}{2} \sec \alpha \times 62\frac{1}{2} \text{ lbs.}$

Its moment about $C = R \times \frac{h \sec \alpha}{3} = \frac{wh^3 \sec^2 \alpha \times 62\frac{1}{2} \text{ lbs.}}{6}$.

For the wicket to be on the point of swinging, the resultant of P and R must pass through B. This resultant $Q' = P - R$, and its value is

$$wH \sec \alpha \times 62\tfrac{1}{2} \text{ lbs.}\left(\frac{H}{2}+d\right) - wh \sec \alpha \times 62\tfrac{1}{2} \text{ lbs.} \times \frac{h}{2} = w \sec \alpha \times 62\tfrac{1}{2} \text{ lbs.}\left(\frac{H^2}{2} + dH - \frac{h^2}{2}\right).$$

Taking moments about C, we have

$$Q' \cdot l' = P \cdot k - R \cdot \frac{h \sec \alpha}{3}, \quad \text{or} \quad l' = \frac{3Pk - Rh \sec \alpha}{3Q'}.$$

Substituting the values of P, k, etc., as before found, we have

$$l' = \frac{\dfrac{wH^2 \sec^2 \alpha (3d+H) \times 62\frac{1}{2} \text{ lbs.}}{2} - \dfrac{wh^3 \sec^2 \alpha \times 62\frac{1}{2} \text{ lbs.}}{2}}{3w \sec \alpha \times 62\frac{1}{2} \text{ lbs.}\left(\dfrac{H^2}{2} + dH - \dfrac{h^2}{2}\right)} = \sec \alpha \times \frac{H^2(H+3d) - h^3}{3(H^2 + 2dH - h^2)}.$$

In order, therefore, to maintain the stability of the wicket the axis of rotation must be above the base a distance equal to or greater than l'.

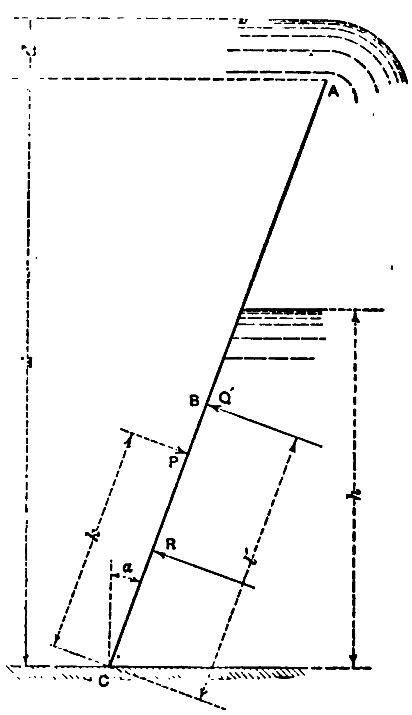


FIG. 216.

In the earliest dams constructed the axis of rotation was placed above the base at or a little over one-third of the length AC. It was found, however, that the wickets were entirely too sensitive, and would swing with very little provocation, lowering the pools suddenly and causing a general disturbance of levels (see p. 585). In the dams of the upper Seine and other rivers, constructed between 1860 and 1870, the axis was accordingly placed at $\frac{48}{100}$ of the length above the sill; in the pass of Port-à-l'Anglais at Paris (1870) it was raised further to $\frac{47}{100}$ of this length; while at the dam of La Mulatière at Lyons (1879) it was placed at $\frac{50}{100}$ of the length. On the weirs of the Kanawha River in this country it is generally located at $\frac{42}{100}$, and on the passes at $\frac{47}{100}$ of the height above the sill. De Lagréné recommends that in navigable passes the axis be placed at one-half the height above the sill, and on weirs at a little more than one-third of the same distance. (See also Pls. 60, 63*a*, and 67.)

Design of Horse. (See Pls. 63*a* and 64.)—On the earlier dams, where maneuvered by a tripping-bar, the horse *BD*, Fig. 214, was inclined slightly upstream, but when placed that it cannot be pulled upstream far enough to pass the vertical and so used with hurters of the Pasqueau type it must be inclined downstream, and be so throw the weight of the wicket to the upstream side. A neglect of this precaution will sometimes cause trouble in lowering the wickets.

The journal-boxes should be bored $\frac{1}{32}$ to $\frac{1}{16}$ inch full, to secure easy movement and prevent rusting up.

The sectional area required in the horse for the tension T and to resist strains from the chain in raising is small compared with that required to stand drift, twisting, etc., and experience is consequently the best guide for proportioning it. The upper crosspiece, to which the head of the prop connects, must be made strong enough to support the bending moment caused by the latter, which equals $\frac{T}{2}\times\frac{f}{2}$, where f = width between the uprights of the horse. This piece is sometimes made with ears forged to and projecting above it, and provided with a pin to which the prop connects. This method, however, is objectionable, as it causes a twisting strain, besides adding to the cost of manufacture. Plates 63*a* and 64 show some of the designs used, but in more recent practice the horses have been made somewhat simpler by reducing the amount of forging, etc. On the Big Sandy dams a built-up horse made of shapes was used. (Pl. 59.)

Design of Prop. (Pls. 63*a* and 64.)—The strain S on the prop is essentially one of compression, and should be determined as for a "pin-bearing" column. A section must first be assumed, and if r = its radius of gyration in inches, and l the length of the prop in feet, the ultimate strength per square inch by the usual formula for columns will be expressed in pounds,

$$\frac{50,000}{1+\frac{(12l)^2}{18,000r^2}}.$$

As with the horse, the section should be made ample. Thus for the Port-à-l'Anglais dam (1870) the strain allowed on the props was only 1600 pounds per square inch.

The head of the prop is bored large, $\frac{1}{8}$ to $\frac{1}{4}$ of an inch, to allow the proper lateral movement when sliding along the hurter.

The lower portions of the props for a pass are usually forged out to a considerable increase of size. The object of this is to provide a weight which the water cannot easily move, and thus secure a proper seating in the hurter. This increased section is usually flattened, but it is believed that a round section would

offer less hold for the current. Experience with props without extra weight at the bottom has not been satisfactory.

The slot shown on the under side of the Ohio props (Pls. 63*a* and 64 and p. 588) was adopted from European practice. Its purpose is to permit the unseating of the prop when the regular means of lowering cannot be used, a hook-pole being employed to catch in the slot and pull the prop sideways. It is stated that this method has proved of value in occasional cases on the Ohio River dams.

The following are examples of sizes used for horses and props:

SIZES OF PROPS.

Location.	Depth on Sill.	Diameter of Prop.	Length of Prop.
Pass of Port-à-l'Anglais, Paris	11′ 10″	3½″	11′ 10″
Passes of Upper Seine	9′ 10″	3½″	8′ 10″
Weirs of Belgian Meuse	About 7′ 5″	3½″	6′ 3″
Passes of Kanawha River, U. S. A.	13′ 0″	3⅝′	12′ 8″
Weirs of Kanawha River, U. S. A.	8′ 6″	3″	7′ 10′,
Passes of Ohio River, 1899, for 6-foot channel depth	13′ 2″	3½″	14′ 7″
Do., 1910, for 9-foot channel depth (Dam No. 37)	About 16′ 0″	4″	15′ 9″

SIZES OF HORSES

Location.	Depth on Sill.	Braces.	Diameter of Journals of Top Arm.	Diameter of Lower Arm.	Uprights.
Pass of Port-à-l'Anglais, Paris, 1870	11′ 10″	Two horizontal	Center, 5″ Ends, 2⅜″	Center, 3⅛″ Ends, 2⅜″	Bar iron, 3⅛″×1⅝″.
Passes of Upper Seine, prior to 1870	9′ 10″	Two horizontal	Ends, 2⅝″	....	Bar iron, 2½″×1½″.
Weirs of Belgian Meuse, about 1876	About 7′ 5″	None	Center, 2 11/32″ Ends, 2⅜″	Ends, 2⅝″	Bar iron, 2⅝″×1⅝″.
Kanawha River, U. S. A., 1896, pass	13′ 0″	Two horizontal	Center, 5½″ Ends, 3″	Center, 2¾″ Ends, 2½″	Two 3″ channels, back to back.
Kanawha River, U. S. A., 1896, weir	8′ 6″	One 1″×2″ diagonal	Center, 2 3/16″ Ends, 2″	Center, 2¼″ Ends, 1¾″	Bar iron, 2″×1½″.
Ohio River, 1899, passes	13′ 2″	Two 3″×¾″ diagonal	Center, 4⅞″ Ends, 2¾″	Center, 2⅞″ Ends, 2⅞″	Bar iron, 3″×1½″.
Do., 1910, pass of Dam No. 37	About 16′	Do.	Center, 5½″ Ends, 2¾″	Center, 3″ Ends, 2⅞″	Do.

Frame of Wicket.—The uprights of the wicket AC (Fig. 217) are subject to a bending moment, the maximum of which occurs at the point of support at B, BA acting as a cantilever. BC is usually made of the same dimensions as required at B; hence, if the part AB is made strong enough to support the maximum moment, BC will possess sufficient strength also.

Let U represent the water pressure on AB, n the distance in feet of its center of pressure from B, m the vertical distance between A and B in feet, and the other letters as assumed in the preceding calculations. Then

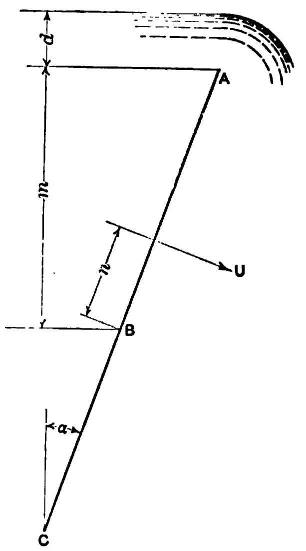


FIG. 217.

$$U = wm \sec \alpha \times 62\tfrac{1}{2} \text{ lbs.} \times \left(\frac{m}{2}+d\right).$$

The distance n, which may be found as shown for the distance k on p. 602, is

$$\frac{m(m+3d) \sec \alpha}{3(m+2d)}.$$

The total bending moment at B, in inch-pounds, is therefore

$$U \times n \times 12 = \frac{wm^2 \sec^2 \alpha}{2} \times 62\tfrac{1}{2} \text{ lbs.} \times \left(d+\frac{m}{3}\right) \times 12.$$

The sections required can then be found, as shown in the calculations for needles (p. 580), by the usual formulas,

$$S = \frac{Mc}{I} \quad \text{and} \quad I = \frac{bt^3}{12}.$$

The following are examples of the sizes used in practice:

SIZES OF WICKETS.

Location.	Depth on Sill.	Width of Wickets.	No. of Uprights.	Width of Uprights.	Bottom Thickness of Uprights.	Thickness of Planking.
Pass of Port-à-l'Anglais	11′ 10″	3.28′	2	12″	8″	2″
Kanawha River, passes	13′ 0″	3′ 8″	2	12″	9″	2″
Kanawha River, weirs	8′ 6″	3′ 9″	2	12″	6″	1½″
Ohio River, passes, 1899, for 6-ft. navigation	13′ 2″	3′ 9″	2	12″	10″	2″
Ohio River, passes, 1910, for 9-ft. navigation (Dam No. 37)	About 16′	3′ 9″	3 to 4	10½″ to 15″	12″	Built solid

Remarks.—The Chanoine dam after some sixty years of trial has proved to be the type best adapted on the whole to rivers subject to quickly rising floods, especially if drift is troublesome, as it is not easily disabled, can be maneuvered rapidly, and does not possess the many loose parts which accompany needle dams and dams of the gate type.* One of the principal advantages of needles and gates

*See pp. 552 and 555 for experiences with ice and drift.

hitherto has been their very small leakage, but under recent improvements the Chanoine wicket can be made as tight as any type of dam in use. (See pp. 571 and 574.) On the other hand it releases a more concentrated body of water during regulation and at times of lowering, and requires a corresponding protection for the foundation. Another objection is that it requires a very wide foundation in order to provide a base for the trestle bridge. Where this bridge can be dispensed with by operating from a boat the cost is much reduced.

An experience of some thirty years with Chanoine dams in the United States has demonstrated their value for rivers with high and rapidly-rising floods, and which carry large quantities of drift, but like most types of movable dams they are at a disadvantage in times of ice, and must be lowered when ice makes its appearance, regardless of inconvenience or injury to traffic.

In Europe few, if any, Chanoine dams have been built within recent years, since the Continental engineers have found other and more economical types to be better suited to their conditions. (See p. 542.)

Time Required for Operation. See page 556.

COST OF CHANOINE WICKET DAMS *

(All have service-bridges except the pass of Dam No. 4, Ohio River.)

Location.	Lift.	Cost per Foot Run.			Remarks.
		Fixed Parts.	Movable Parts.	Total.	
Marne	6′ 6″	....	....	$244.00	Cost of passes.
Saône	7′ 6″	....	....	244.00	Cost of passes.
Belgian Meuse	8′ 3″	$75.00	$78.00	153.00	Cost of weirs.
La Mulatière, Lyons	11′ 10″ (max.)	508.50	91.50	600.00	Cost of pass.
Kanawha River, W. Va., Dam No. 7, 1893	8′ 0″ (13′ on pass sill)	226.00	26.80	252.80	Average of pass and weir.
Ohio River, Dam No. 4 (1906), Pass.	16′ 9″ (on sill)	210.00	61.00	271.00	Wickets, 17′ 10″ long.
Do. Do. Weir.	....	256.00	72.00	328.00	Cost of abutment ($37,000) not included.

* The total costs of the Kanawha and Ohio River locks and dams will be found in the tables at the end of the book.

CHAPTER VIII.

GATE AND CURTAIN DAMS: BRIDGE AND SHUTTER DAMS.

GATE DAMS. (See also Bridge Dams, pp. 623 to 639.)

General.—The gate dam or Boulé gate dam, as it is often called, an evolution of the old stop-plank sluices, was first used by M. Boulé, the French engineer, at the Port-à-l'Anglais dam near Paris in 1875, and has since been applied in France, Russia, America, and elsewhere. It is a modification of the needle dam, the vertical needles being replaced by horizontal planks, the ends of which rest against and slide upon the faces of trestles, or upon uprights supported from a bridge. Where trestles are used they are hinged to the floor and are raised and lowered in a manner similar to trestles for needles. (See Fig. 218, Pl. 68, and illustrations in this chapter.)

Maneuvers.—To raise the dam, the trestles are first set upright, and connected by the floors and service-track, and the gates are brought out on a truck and set in place by a light movable crane, or, where small enough, by a pole with a hook on its end. As the gates are put in place the head of course increases and the sliding friction becomes such that it counteracts the weight of the gate; recourse is then had to a long spar moving in the plane of the gate and fitted with a cast-iron rack worked by a winch, the whole being placed on a car, which is anchored to the trestles when being used. The end of the spar is fitted with a cross-piece which is placed on the top of the gate and is of similar width; the winch is then turned, and the spar forces the gate down into place. The apparatus is also used to raise the gates against a head, the end of the spar being provided with a hook which engages with an iron handle on the gate, or a simple chain with a hook may be used instead. (See p. 611.) The machine is effective but cumbersome, and the introduction of rollers referred to further on has proved a great improvement, as comparatively little power is needed where they are used, and the spar and winch can often be dispensed with. As a rule the gates of the second or third row are the hardest to operate, since by the time these are removed the lower pool has risen and reduced the head on the gates below. (See also "Remarks," p. 615.) The regulation of the pool is effected by maneuvering the top row of gates, which are usually made from 10 to 12 inches in width, to permit easy movement. The lowering of the dam is done by reversing the operation of raising.

Details of Construction, etc.—The trestles for this type of dam have usually been spaced from $3\frac{1}{4}$ feet (one meter) to 4 feet apart, with a service-bridge 9 to 18 inches above the pool, and are considerably heavier than those for needle dams. Their upstream face is made smooth, and provided with a projecting rib, serving as a guide for the gates when moving. This rib need not be more than an inch or two in height and should not project beyond the face of the gates, or it will interfere with the placing of the vertical wooden strips or "joint covers" which are used in low water over the ends of adjacent rows of gates to stop leakage. These ends, which increase in thickness from the top to the bottom rows, are

FIG. 218.—Perspective View of Boulé Gates and Trestles, as Used at the Suresnes Dam.

trimmed off to make a continuous surface against which the joint cover rests. (See Pl. 68.)

The gates are usually made of the same height—from 3 to 4 feet—for all rows except the top one, which varies from 4 to 21 inches, 12 inches being a convenient height for it. Where trestles are used the maximum area of any gate rarely exceeds 15 to 18 square feet. The gates are generally made of wood strapped with iron and provided with a handle for maneuvering (see Fig. 218). The first iron ones used were on the Pretzien dam on the Elbe (1875), and later examples with gates of a considerable size are to be found on the bridge dams on the Oise (1901), on the bridge dam at Mirowitz in Bohemia (1903), and elsewhere.

On the pass of the Libschitz dam (Bohemia, 1900, see Pl. 68 and accompanying illustrations), with 14 feet 9 inches on the sill, the Boulé gates used were five in number for each bay, four being $3\frac{1}{4}$ feet high, and the top one 1 foot 9 inches high. The planking of the lowest one was 5 inches thick. The trestles were spaced $1\frac{1}{4}$ meters apart, and were made 19 feet 8 inches high, weighing about 3000 pounds each. The recess behind the sill had to be $3\frac{1}{4}$ feet deep.

On the Boulé dam at the head of the Louisville and Portland Canal, Kentucky, around the falls of the Ohio, built in 1899 and the first dam of this type in this country, the trestles were spaced 4 feet apart, and were 7 feet high, being composed of 1 by 4 inch bar iron. The depth of the sill was 5 feet, and the lowest gate was 2 inches thick. The dam was built with three openings, each 200 feet long, and one opening about 50 feet long. It is used as a flushing weir for the basin at the head of the canal. A somewhat similar construction was used later for a portion of the dam built in the vicinity across the Ohio River. Examples of Boulé gate crests on fixed dams have been described on p. 522.

The first trestle Boulé dam in the United States approaching the height of the European dams was built on the Trinity River in Texas, a few miles below Dallas (Pl. 69). It was completed in 1909. As the river is very narrow at the location it was decided to use a single opening, and for financial reasons it was necessary to make it as short as consistent with retaining the natural area of discharge. The opening was accordingly made 84 feet long with a depth of 16 feet on the sill (the lift being $11\frac{1}{2}$ feet), a drift chute 15 feet long being placed in addition next to the lock wall. This depth being too great for easy regulation by needles, it was decided to use a Boulé dam, as this type had proved suitable to regulation with considerable depths and was also economical in first cost and permitted very little leakage. The trestles were made about 23 feet in height and spaced 4 feet centers, with a walkway about 4 feet above the upper pool. Regulation is done (except for the effect of the drift-chute) through this single opening, as at the dams of Suresnes and Mirowitz (pp. 618 and 628), and the maneuvers of the gates are performed chiefly by a steam derrick-boat, as the method of the operating spar (p. 608) was found to be too slow. Seven tiers of gates are used, varying from $2\frac{7}{8}$ inches in thickness and 2 feet $11\frac{1}{4}$ inches in height for the four lower tiers to 1 inch in thickness and 1 foot 3 inches in height for the top tier. All tiers except the top one have ball rollers. (See p. 613.) Drift has caused a good deal of trouble at this dam, as the rises are very rapid and bring large quantities of débris with them almost at once, and in spite of the shortness of the opening, which it was hoped would alleviate this danger, it is difficult to remove a sufficient number of the parts in time to prevent clogging. The structure will be remodeled, using Chanoine wickets.

A description of some of the European dams with Boulé gates, of the original or of a modified type, will be found in the succeeding paragraphs on "Curtain

GENERAL VIEW OF THE LIBSCHITZ DAM, BOHEMIA, IN OPERATION, SHOWING THE METHOD OF HANDING THE BOULÉ GATES BY A TRAVELING HAND-CRANE.

The view is taken from the lock, and shows the pass in the foreground, with the weir and the raft chute on the other side. The pass is 213 feet long, with 14.7 feet on the sill, and is closed with Boulé gates. The weir is 160 feet long, with 10.4 feet on the sill, and is closed with needles. The lift is 12.7 feet. The work was completed in 1900. (See also pp. 619 and 620 and Pl. 68..)

VIEW SHOWING THE LOWERING OF THE PASS TRESTLES, LIBSCHITZ DAM, BOHEMIA.

(See also p. 619 and Pl. 68.)

Dams" and "Bridge Dams" (pp. 616 and 623 and after), and dimensions of trestles are given on p. 622.

Use of Rollers.—In 1895 experiments were begun at the Marolles dam in France with gates provided with rollers, in order to find some means of reducing the objectionable sliding friction, which becomes in some cases as much as 2000 pounds per gate. Ordinary rollers were tried and also ball-bearing rollers, the latter proving the more satisfactory. These experiments showed that one man could easily raise a ball-roller gate of 5.8 square feet area against a head of 7 feet, using only a hook-pole.* The arrangement proved of so much advantage that after a few seasons' trial it was introduced elsewhere, and the use of sliding gates, except for the top rows, became practically abandoned for new dams. Tests made in the United States with gates 5 feet long by 2 feet high, of design as shown on Pl. 69, each with a total load of 13,000 pounds, and with the rollers moving on iron plates so as to give the same conditions as when bearing against a trestle, gave a maximum pull to start the gate of about 400 pounds, and about 300 pounds to keep it moving.

The general design of a gate with ball-rollers is shown on Pl. 69. Side rollers are also used in some cases to prevent the gate binding between the slides when moving, but their necessity is questioned by many engineers. It has been found by experience that the rollers must be cleaned with a hose or other means about every month or six weeks, one gate being removed at a time and a spare one substituted. If this is not done, grit and silt will collect in the pockets and gradually clog the roller until it refuses to turn. For similar reasons no oil or grease must be put in, as this will only serve to catch the grit. In designing the parts therefore space should be left to facilitate the cleaning.

Stoney Gates and Rollers.—A British engineer, Mr. Stoney, invented the Stoney roller, which gives the name to the "Stoney Gate," the gate itself being merely a Boulé or sluice gate, though usually of large size. The rollers are not attached to the gate, but move freely (just like the expansion rollers under a bridge) their ends being held by bars or frames which are secured at the top by a chain and counterweight (Fig. 219). The gate moves over these rollers which in turn move over a cast-iron plate bolted to the masonry, and travel one-half the movement of the gate. When the gate is counterbalanced, the moving friction is theoretically done away with, except in the gearing. In point of fact, however, there is usually a good deal of friction, due to rusting, to imperfections in manufacture and erection, and to the small size of the rollers making obstructions of sand, etc., difficult to pass. Experiments on the Chicago Drainage Canal with the Stoney gates, which had been very rarely operated, showed that the friction to be overcome,

* Annales des Ponts et Chaussées, April, 1896. These roller gates have been in use at Marolles since 1895 and at St. Mammès since 1897 with perfect satisfaction.

including that of the operating machinery, was about 0.04 of the water pressure on the gate, while tests for the Panama Canal sluices with gates placed horizontally and suitably loaded with 270 tons showed a starting friction equal to about 0.018 of the total load, and a moving friction equal to about 0.013.

Some very large gates of this design have been successfully built, among the largest being those at St. Mary's Falls, Michigan, measuring about 60 feet in length by 25 feet in height, and operated under a head of 6 to 10 feet. At the Assouan dam on the Nile (described on p. 305 and after), some of the Stoney gates originally sustained a head (measured from the sill) of about 87 feet at low water, the maximum head under which discharge was made being about 55 feet. These figures refer to the dam as originally built, and before the crest was raised.

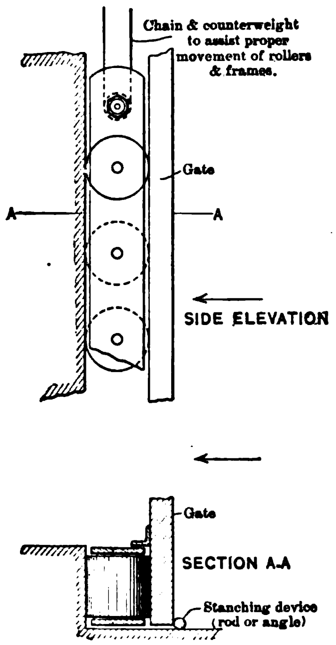


FIG. 219.—Sketch of Stoney Rollers.

The formula usually employed for proportioning steel rollers of the Stoney type is similar to that for bridge-seat rollers, and is:

$$\text{Allowable pressure per lineal inch of length of roller} = 600d,$$

where d = diameter of roller in inches. For the sluices of the Assouan dam the rollers on some of the gates were made about 12 inches long and 8 inches diameter, and placed close together, and on the Mirowitz dam, with a normal head of $12\frac{3}{4}$ feet, they were 3 inches long and $3\frac{1}{4}$ inches diameter, placed from 13 to 17 inches center to center. The last-named dam is described on pp. 628 to 631.

In sediment-bearing streams it has been found that as the gate ascends and leaves the lower rollers exposed to the current, there is a decided tendency for the traveling sand and gravel to wear away their faces.

Balanced Gate. An automatic gate or shutter weir of unusual type is to be found on the Grafenauer Ohe in Bavaria, where it is used to regulate the level for a paper mill. It consists of an ordinary steel sluice gate about 24 feet long and 6.8 feet high, hinged at the bottom so that it will lie flat on its masonry sill, like a Thénard shutter. Each upper corner of the gate is fastened by a cable to an overhead cylinder placed above flood level and arranged to roll on an inclined track. The cables are wound around the ends of the cylinder, and as the water forces down the gate the cables pull on the cylinder and roll it up its track, the two main parts being adjusted so that the loads approximately balance. As the water falls and reduces the pressure on the gate, the cylinder rolls back, winding up the cables and pulling up the gate until the loads balance again. The

cylinder is 1.7 feet in diameter at its ends and 2.9 feet at the center. The dam is said to work very satisfactorily.*

Revolving Gates.—On the River Spree, in the city of Berlin, is a structure of much interest, known as the "Mühlendamm," and completed in 1893. It consists of a dam and lock, located near and under the crossing of two streets. The lock is of ordinary construction, $31\frac{1}{2}$ feet wide and 377.2 feet between hollow quoins, with a depth of $6\frac{1}{2}$ feet on the sills and operated by hydraulic power obtained from an accumulator and an automatic 25 horse-power turbine coupled to pumps. The dam is of the bridge type, and consists of three openings of about 50 feet each center to center of piers. To each bridge is permanently attached a series of vertical iron uprights (Fig. 220) spaced $8\frac{1}{2}$ feet apart, which support iron gates about $8\frac{1}{4}$ feet wide and 16 feet high, moving on rollers. The upper ends of the uprights are curved, so that when a gate is hoisted it begins to revolve and finally assumes an almost horizontal position. This arrangement saves headroom and reduces the height of the piers, etc. The operation is done by a long shaft and chains, power being supplied by the accumulator referred to above.

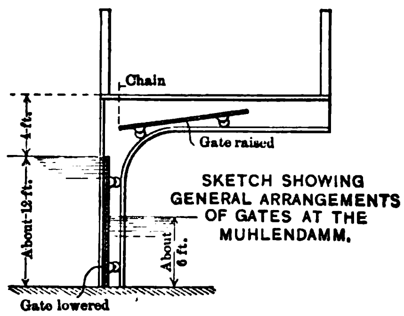


FIG. 220.

The dam across the Thames at Richmond, near London, built in 1894, is composed of three large gates of the Stoney type, each about 66 feet long by 11 to 12 feet high, sustaining ordinarily a maximum head of about 8 feet. They are counterbalanced and operated by winches on an overhead bridge, the operating shaft being as long as the gate. These gates possess a novel feature in that they commence to revolve soon after they clear the water surface, and when fully raised lie horizontally under the bridge. The turning movement is done by a cranked arm at each end of the gate, placed at the center of gravity, at which point the hoisting cables are attached. The projecting end of the arm travels in a slot curved at its upper end, so that when the gate reaches the proper height the arm meets the curve and is gradually forced over into a horizontal plane, turning the gate with it. The gates are usually raised at every tide and have proved very satisfactory. The method saves headroom and avoids the unsightly appearance of a large gate suspended vertically.

Remarks.—The Boulé gate has proved under suitable conditions to be an excellent device, as it is simple, and with proper appliances can be maneuvered under the full head of water. It is one of the two types (the other being the needle dam) considered by European engineers to be especially suitable to their rivers. (See p. 542.) It is also cheap in construction, as it needs no great width of foun-

* "Engineering News," December 29, 1910.

dation, having in this respect an advantage over the Chanoine dam, which is almost always provided with the service-bridge for operating, requiring a special width of substructure. Comparative estimates of the two types for the dam at Port-à-l'Anglais, on the Seine, showed a difference of 30 per cent in favor of the Boulé gate. These gates also minimize the dangers of scour, since the discharge is usually made from the surface of the pool, but in practice this advantage is not of great importance, as the amount of protection necessary to withstand regulation during rises does not vary much with different types of dams under similar conditions. Another advantage is that the Boulé dam is very tight, as the joints are few and close and become quickly clogged with leaves and other débris in suspension. In this respect it is in a measure superior to the needle dam (except where wide needles are used) as there is no warping of the pieces and there are fewer open joints.

The principal objection to the old type of Boulé dam, where sliding gates were used, was that the operation was comparatively slow, each gate having to be handled separately and a good deal of power being required. To remove a gate from the second row below the surface took from 1 to 2 minutes, and to remove one from the bottom row, 5 or 6 minutes. For this reason the type was not well adapted to rivers of quick rises, except where the dams were short. The introduction of rollers, however, described on p. 613, largely removed this objection, as they rendered maneuvers comparatively easy and tended to place the Boulé dam on a par with the needle type as regards simplicity of construction and operation. Both types, however, have the objection of requiring the handling and care of numerous pieces, often at a time when every minute is of importance.

Where used in the United States for weirs or as movable tops on fixed dams the Boulé gate has in general given satisfaction, but the experience on the Trinity River (p. 610) has shown that where the dam is of any height and is exposed to sudden rises and quick runs of drift it is difficult to remove the pieces rapidly enough to prevent damage. This danger is one from which the dams of this type in Europe are largely free.

Time Required for Operation, see p. 557.

CURTAIN DAMS. (See also Bridge Dams, pp. 623 to 639.)

General.—The Caméré curtain was the invention of the French engineer M. Caméré. It consists of narrow horizontal strips of wood hinged together, and capable of being rolled up by an endless chain which passes round them, each curtain reaching from the surface of the water to the sill. (See Figs. 221 and 224, also Pls. 70 and 71.)

This type was introduced at the trestle dams of Suresnes and Villez and at the bridge dams of Meulan, Méricourt, Port-Mort and Poses (all on the lower Seine);

at the outlet of Lake Geneva; on the Oder, and elsewhere, and in general has given excellent satisfaction.

Maneuvers.—The curtains are supported by trestles, of design similar to those for Boulé gates and similarly raised and lowered, or by frames suspended from an overhead bridge and resting against shoes. When the dam is to be closed the curtains, each of which is attached to a rectangular iron frame, are brought out on a truck, and the frames are set in place on the top of the trestles by a special winch and unrolled by releasing the chain (Fig. 221). The pool is regulated by rolling them up from the bottom as much as may be desired. When the dam is to be opened they are rolled up entirely, and the frames with the curtains attached are lifted off, placed on the truck, and taken to the storehouse. When bridges are used instead of trestles, the curtains are often left attached to the heads of the uprights.

FIG. 221.—Method of Handling Curtains with a Service Truck and Crab.

Remarks.—The Caméré curtain possesses an advantage over the sliding Boulé gate in that it does away with the friction of the latter, and hence can be used for higher lifts, that of the Poses dam being 13.7 feet, with 16.4 feet of water on the sill of the deepest pass. It also permits a very exact regulation of the pool, and a speedy one where everything works smoothly. On the other hand, the curtain is considerably more expensive than the gate, and much more liable to get out of order, owing to its more complicated parts. At the Suresnes dam, described in the next paragraphs, gates and curtains were used side by side for the purpose of thoroughly testing their comparative merits. We were informed in 1905 by those who had been connected with their operation since 1885 that they considered the Boulé type the superior. Their experience had showed that while the curtain had many apparent advantages, and could be operated with a saving of 25 per cent in time as compared with the gates (which were all of the sliding type without rollers), they were complicated and fragile; sticks and small débris would catch between

the strips and necessitate the unrolling and cleaning of the curtains after they had been taken ashore, and lastly they required very careful handling in rolling or unrolling, as they were liable to become wedged between the trestles or against each other. For simplicity and strength, and general capacity to withstand handling, the engineers' verdict was in favor of the gates. For these reasons no dams of the curtain type have been constructed in Europe within recent years.

Additional details will be found in the description of the Poses Dam, pp. 626 to 628.

The Suresnes Dam.—The Suresnes dam (see Fig. 218, p. 609), located just below Paris, was built by M. Boulé in 1885, and was founded on bed of substantial clay. It is one of the largest works on the Lower Seine, and one of the best examples of movable dams to be found. It was constructed with alternate bays of Boulé gates and Caméré curtains. The trestles of the pass are $19\frac{1}{2}$ feet high, and the 6 rows of gates are about 17 feet in total vertical height (each row being about $3\frac{1}{2}$ feet high measured on the slope, except the top one of about 1 foot) supporting a head of 10 feet 8 inches. The planking varies in thickness from $1\frac{5}{8}$ inches for the second row to $3\frac{1}{4}$ inches for the bottom row. The dam is divided into three parts by two islands. The navigable pass was made about 238 feet wide, and located in the left arm of the river, while in the right arm there was placed an elevated pass about 203 feet wide. Between the two islands there was placed a weir also about 203 feet wide. The sills of these three passes are 15.0, 13.3 and 10.0 feet, respectively, below the level of the upper pool, and the masonry sloped down towards the trestles, making the bottom of the gates considerably below the sill level. This was done to reduce any tendency towards sanding-up.

The gates and curtains are supported by trestles 19.5, 17.9, and 13.44 feet high, weighing respectively 4000, 3000, and 1760 pounds, and all spaced about 4.1 feet centers. They were constructed of channel iron, with two uprights front and back, joined by cross-bars and braced by a St. Andrew's cross. A continuous chain is used to raise and lower the trestles, operated by a crab on the shore. This method allows them to be maneuvered six at a time. The diameter of the chain iron for the navigable pass is about 1 inch, the average pull when raising being about 4500 pounds. The crab, however, was given a power of 10 tons.

In the right arm of the river, i.e., in the elevated pass, Caméré curtains were exclusively used for closing the dam. The middle channel or weir is closed with Boulé gates, and the left arm or navigable pass with gates and curtains alternately. The last method of arrangement has proved the best, for the reason that the curtains when rolled up do not always preserve a cylindrical form. This is liable to cause adjacent curtains to become wedged during operation, and if delays ensue, regulation can be carried on by the gates.

When the river rises the gates of the navigable pass are first removed, and if the flood increases, the curtains are rolled up and the trestles in the navigable pass are

VIEW SHOWING TRESTLES FOR BOULÉ GATES AT THE LIBSCHITZ DAM, BOHEMIA, TAKEN DURING CONSTRUCTION.

The trestles are 4 feet 1 inch apart and 19.7 feet high, weighing, with floors and all attachments, 3740 pounds each. The clamp for the operating chain is seen on the top member. (See also pp. 611 and 612 and Pl. 68.)

VIEW OF THE RAFT CHUTE AT THE LIBSCHITZ DAM, WITH A RAFT PASSING THROUGH. (See also p. 611.)

lowered. Thus the whole pass is opened to navigation before much regulation is done on the weirs. As it involves a considerable delay to pass boats through the locks, the method adopted is to concentrate the discharge in the navigable pass as far as practicable, and to lower the trestles as soon as there is sufficient water on the sill ($10\frac{1}{2}$ feet minimum) for the purpose of navigation. If the river falls, the trestles and some gates are immediately replaced in order to preserve a proper stage of water. In the other passes, which are but little used for navigation, the trestles are not lowered until almost submerged. The curtains are removed by means of a special car carrying a small windlass, and are carried to a platform on shore and hung upon a special frame in the same position as the dam. The operation is laborious, as each curtain and frame weighs 1600 pounds. As they are rolled up, débris of various kinds catches between their sticks, and they have to be unrolled and cleaned on shore. The gates, which in 1905 were all of the sliding type, are operated by the spar and winch described on p. 608, and when removed are placed on a truck rolling on the rails of a foot-bridge, when they are taken to the bank and stacked there. The time required to lower the trestles of the pass (fifty-seven in number) is three hours, and to raise them five hours, the latter operation requiring six or seven men. (See also p. 557.)

A new lock, 525 feet long and 56 feet wide, was placed between the left bank and the old lock. The latter was rebuilt, and at its lower end a new miter wall and gates were placed. A new chamber 165 feet long was then added.

The cost of the dam is given as $632 per lineal foot, all included. (See also p. 639.)

Calculations for Trestles for Gates or Curtains.—Trestles for use with the Boulé or Caméré types of dams have to support, in addition to the direct strains, the bending from the water-loads on the upstream leg.

To determine the dimensions required, first find by moments or by graphics the strains in the various pieces AB, AG, etc. (Fig. 222), induced by the water-loads P_1, P_2, etc., considered as concentrated at the panel points A, B, etc. This will give the maximum strains in the web and downstream members, but the upstream member $ABCD$ has to support in addition, between each panel point, the bending from the gates or curtains, which rest directly on it. In all ordinary cases economical manufacture will require $ABCD$ to have the same section from top to bottom, as, for instance, two channels and a cover-plate, so that only the maximum bending moment, which occurs in CD, need be found. One-fourth may be deducted from this moment, as AD acts as a continuous beam, and the result used in the formula below.

The total area of section required will then be

$$A=\frac{1}{S}\left\{d+\frac{Mc}{r^2}\right\},$$

where A = total area required; S = total allowed strain per square inch; d = amount of direct compression or tension in the member; M = maximum bending moment in inch-pounds; c = distance in inches of extreme fiber from neutral axis; r = radius of gyration of the section.

The gates or curtains themselves have to sustain the pressure of the water, which equals for any gate

$$w \times h \times d \times 62\tfrac{1}{2} \text{ lbs.} = P,$$

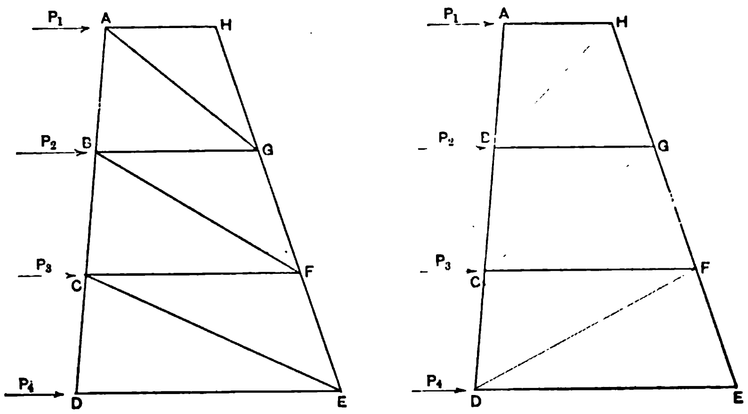


FIG. 222.

where w = horizontal length of gate, h = its height, and d = distance from surface of water to center of gate, all in feet.

The bending moment is $\dfrac{P \times w}{8} \times 12$ inch-pounds $= M$, and $M = \dfrac{SI}{C}$, where S = extreme fiber stress per square inch, C = half the thickness of the gate, and I = its moment of inertia.

DIMENSIONS OF TRESTLES FOR GATE AND CURTAIN DAMS

Location.	Height.	Width of Base.	Ratio of Base to Height.	Distance, C. to C.	Lift.	Weight, Each, Pounds.	Remarks.
Villez, Lower Seine (1883–5)	17′ 9″	14′ 9″	$\frac{83}{100}$	......	9′ 10″	4360	Curtain dam.
Port-à-l'Anglais, Upper Seine..................	17′ 2″	10′ 2″	$\frac{59}{100}$	......	8′ 9″	2120	Gate dam.
Suresnes, Lower Seine (1885) See p. 618..............	19′ 9″	12′ 4″	$\frac{63}{100}$	4′ 1½″	10′ 8″ (15′ 0″ on sill)	4000	Gates and curtains.
Libschitz, Bohemia, Moldau River (1900) See Pl. 68 and p. 619..............	18′ 11″	12′ 6″	$\frac{66}{100}$	4′ 1½″	12′ 10″ (14′ 8″ on sill)	3750	Gate dam. Has two floors, side by side, each 30″ wide and 1′ 8″ above pool. See p. 611.
Trinity River, Texas, Dam No. 1 (1907). See Pl. 69.	22′ 11″	12′ 9″	$\frac{55}{100}$	4′ 0″	11′ 6″ (16′ 0″ on sill)	2400	Gate dam. Floor 2′ 10″ wide c. to c. of car-rails and 4′ 0″ above pool.

Time Required for Operation.—See p. 557.

Cost of Gate and Curtain Dams. See p. 639.

BRIDGE DAMS.

General.—The bridge dam differs from the gate and curtain dams just described in that the closure is not supported by trestles, but by uprights or frames hinged to a bridge above. In some cases the closure has consisted of curtains, as on the dams at Poses and Port Mort on the Lower Seine; in other cases gates have been used, as at the Nussdorf dam at the head of the Danube Canal in Vienna, at Mirowitz on the Moldau in Bohemia, and on the Mohawk River in New York State. Although the actual construction of dams supported by bridges is of comparatively recent date, M. Frimot proposed such an arrangement in 1829, and a bridge sluice dam closed by needles was constructed on the Upper Yonne as far back as 1836 (Fig. 223). These designs, however, did not provide for navigation under the arches, except when the dam was open and the water low. Later, on the river Elbe, at Pretzien in Prussia, a bridge dam was built in 1874–75, the closing being made with sliding gates. It was built with nine bays each 41 feet

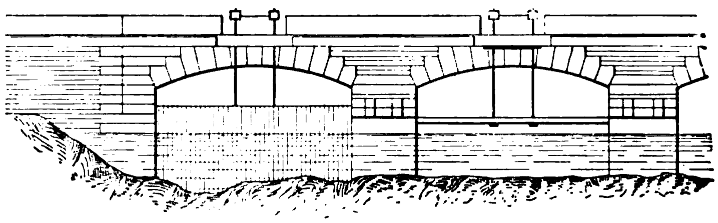


FIG. 223.—Bridge Dam on the Yonne, France. 1836.

wide, separated by piers on which the bridge rested. The clearance, however, was too low to permit boats to pass underneath, even with the dam open. The floor of the dam was above low-water level and the pool level was 10 feet above the sill. This structure was followed between 1880 and 1885 by the construction of four large bridge dams in the lower Seine at Meulan, Méricourt, Port Mort, and Poses, as the unprecedented depth of water required for the navigation, $10\frac{1}{2}$ feet, necessitated in some of these cases unusual lifts and apparatus. These were the first bridge dams built in connection with an open navigation during floods, and they showed so many advantages where high lifts and great depths on the sills could be used that several very important structures of this type have been built since.

Maneuvers.—The maneuvers are conducted on the same general principles as with trestle dams, except that the gates or curtains forming the closure are usually left upon the uprights or frames instead of being carried ashore (see Pl. 70). To put the dam in position the hoisting chains of the uprights are disconnected from the latches on the upstream side of the bridge, and the uprights—which are usually braced together in separate pairs—are lowered one pair at a time until their ends rest against the cast-iron shoes built into the masonry foundation. The other ends

are permanently fastened to the downstream side of the bridge by ordinary steel pins, on which they turn as on a hinge. When all the uprights are in place the gates or curtains are lowered (as with trestles) and close the dam, and if the supply of water becomes low, wooden joint covers are placed so as to close the spaces between adjacent uprights. Regulation is done as before described, by raising part of the closure, and the removal of the dam is the reverse of placing it: the gates or curtains are pulled to the top of each pair of uprights, and the latter are hoisted out of the water and hooked up under the bridge. At the dam at Mirowitz a single gate moving on rollers and long enough to reach from the surface to the sill was adopted, thus providing a regulation similar to that with Caméré curtains, but without their complications. (See p. 628.)

Details of Construction, etc.—The uprights were at first spaced about 4 feet between centers of pairs with curtains of corresponding width, but at the Poses dam (1885), where the uprights were spaced about 4 feet, the curtains were made about 7½ feet long, resting upon two pairs of uprights, which were fastened and hoisted together (see Pl. 70 and p. 628 at top). This was done so that if the maneuvering of so wide a curtain gave trouble, they could be changed and worked with single pairs. As no trouble was experienced, however, the intermediate uprights were omitted at the Mirowitz dam (1903), and the closure was composed of iron gates about 6 feet wide and 17½ feet high, moving on Stoney rollers and resting against and between a single pair of uprights spaced 7.35 feet center to center of pairs. On the Mohawk River bridge dams for the New York State Barge Canal (1907 and after) the uprights were spaced 15 feet center to center of pairs and the gates made about 30 feet long, overhanging about 7½ feet at each end (see Pl. 72 and pp. 633 to 635). The bottoms of the gates when fully hoisted are about 2 feet above the elevation of the upper pool.

Time Required for Operation.—See p. 557 and after.

Remarks.—The principal advantages of the bridge dam are, that it permits safe and easy operation, the dam-tenders not being exposed to the various risks or to the amount of labor incurred with trestle dams, such as the many pieces to handle and carry ashore, the catching of drift in the ironwork which may delay or prevent the lowering of part of the dam, etc.; that there is no ironwork under water when the dam is not in use (except the cast shoes) to rust or become injured by sunken drift or branches of passing trees, all parts being where they can be inspected and preserved; that the parts can be made few and strong, and therefore more able to withstand wear and accidents, and requiring less detail in operation; that the regulation of the pool is easily carried out; and that greater lifts and depths on the sills can be used than might be safe with trestles. The effects of ice are referred to on p. 552. The experience of twenty years with this type of dam on the Lower Seine was sufficiently favorable to lead to the decision in 1905 to replace the

needle dam at Martot on that river, at the head of tide-water, with a bridge dam by gates of design generally similar to those at Mirowitz (see p. 628), while at this latter dam, built with a bridge-and-gate pass in the center flanked by a needle weir on each side, all regulation has been carried on by the pass. The original intention was to regulate by the weirs, opening the pass only in floods, but the latter was found so much more convenient for this purpose that the operations have been reversed, and the needle weirs are not maneuvered except when high water necessitates the removal of the entire dam.

The objections to the bridge dam are that the width of opening must be moderate for reasons of economy, as the weight of the bridge increases approximately as the square of the span (spans from 100 to 200 feet are about the limits used, the maximum being two spans each of 240 feet clear width at Dam No. 8 on the Mohawk River, N. Y., this length being employed because of local conditions of ice gorges); that the bridge limits the headroom during floods unless special devices are employed, as on the Oise dams (1901), where a low-level bridge is used to support the uprights and is suspended from a high-level bridge up to which it is hoisted when the dam proper has been removed (see Pl. 74); that the whole construction is expensive, and that much power is needed for operation. The last objection is not of special weight, as nearly all modern dams of the movable type, unless small, have to be fitted with operating power, if only that of hand winches, and the amount needed for a bridge dam is by no means excessive in proportion to its lift. As regards the expense, the bridge dam is rarely economical except for high lifts or great depths on the sills, and when these conditions have to be met it may prove to be the cheapest type. Comparative estimates were made in 1905 between bridge-and-gate dams and trestle-and-gate dams for the Mohawk River canalization of the New York State Barge Canal. The lift in each case was assumed as about 14 feet and the depth on the sill as 17 feet. The uprights for the bridges were taken as 15 feet centers, and the trestles as 8 feet centers, and the estimates showed a saving of not more than 10 to 15 per cent in favor of trestles. Where a greater depth on the sill was used, the estimated saving was considerably less, and the bridge dam would have the advantage of being certain in operation, while wide-span trestles of high lift would be experimental. The former type was therefore chosen, and contracts were made for construction within the estimated cost.

Another objection to the standard type of bridge dam is that the uprights supporting the gates must be hoisted upstream or against the current, and that if ice or drift should pack against them it might not be possible to move them. Although this objection has more weight in theory than in practice, since cases where such conditions have given any trouble have been extremely rare, it undoubtedly exists, and a design to overcome it was used in rebuilding a power dam at Bitterfeld on the Mulde, a tributary of the Elbe, in 1900. The dam replaced was

composed of wooden gates sliding on removable wooden uprights or posts supported at the top by a service bridge. This dam had been caught by a sudden flood during harvest time, and large quantities of hay, drift and other débris had caught around the gates and uprights, preventing their removal and resulting in extensive damages (see also p. 552).

The new dam was made of the same dimensions as the old one, and has a total length of opening of 185 feet, divided by masonry piers into five bays each 35 feet wide. These are subdivided by hinged uprights into five panels each about 7 feet $4\frac{1}{2}$ inches wide, on which move iron gates, each 7 feet high and about 7 feet long. There is only one row of gates and there are four rollers to a gate, fixed to its downstream side. Each gate weighs about 1100 pounds and is hoisted by chains operated by worm-gear hand-winches. The uprights are hinged to the upstream side of a girder bridge and are operated by a cast gear-quadrant which is bolted to the upright where it extends above the bridge. (Pl. 74.) The feet of the uprights are held by cams, and when these are released the uprights swing downstream and can then be hoisted under the bridge. The uprights in each bay are fastened together at top and bottom and thus move as a unit. They are operated by hand gear connected to the quadrants. The rollers of the gates move between the channel flanges of the uprights and thus the gates do not fall out when the uprights move from the vertical. It is stated that the uprights, when the cams are unlocked, are released practically as a unit and without jerking. The gearing at the top of course assists in preventing any uneven movement.*

In 1908 a dam on similar principles was designed for the Weser, with bays about 100 feet long, subdivided by uprights carrying gates about 13 feet in length. The maximum head was to be $11\frac{1}{2}$ feet.

On one of the weirs at Kvarnsveden on the River Dalelf in Sweden (built about 1907) is a bridge portion where the uprights are arranged to swing sideways and are pulled up under the bridge something like inverted trestles. Closure is made with Boulé gates. This weir was built for power purposes and is in use all the year.†

Details of some of the principal bridge dams are appended.

Poses Dam.—This dam is on the Lower Seine, the second one from tide water, and is one of the largest movable dams in existence, besides having held for some twenty years (until the completion of Needle Dam No. 1, Big Sandy River, described on p. 569 and after) the highest lift of this class of structure. It was completed in 1885, forming a link in the system which provided a least depth of $10\frac{1}{2}$ feet between Paris and the sea. The dam was founded on a bed of chalk and was made of

* For plans, etc., see Proceedings 11th Int. Cong. Navigation, 1908, paper by Mm. Schnapp and Carstanjen. It was learned on inquiry in 1910 that the dam had given no cause for complaint during its existence. A small dam of similar principles was built in 1898 on the Aa near Bocholt, and a few other examples are to be found.

† See paper by Messrs. Hansen and Malm, Int. Cong. Navigation, 1908.

the pure bridge type (no highway being provided), and is closed by curtains. The lift was made 13.7 feet, and the distance between abutments $771\frac{1}{2}$ feet, the net opening being about $693\frac{1}{2}$ lineal feet. This was divided into two weirs with 9.8 feet on the sills and 99 feet long each, three passes with 16.4 feet on the sills and $96\frac{1}{2}$ feet long each, and two passes with 16.4 feet on the sills, each 103 feet in length. The two passes next the left bank were provided with high bridges, giving a clearance of 17.2 feet above the highest flood.

A reference to Figs. 224 and Pls. 70 and 71 will show that two lines of bridges were provided and at different levels, for convenience in construction and operation.

FIG. 224.—General Section of the Poses Dam. (Built on a chalk foundation.)

The girders of the bridges for the two navigation passes were made continuous and extend over both openings, expansion being provided for at each end and the center being fixed, and those over the five remaining openings were similarly constructed, forming continuous girders more than 500 feet long. Vertical rollers are provided at the piers to take care of the horizontal water thrust. The frames or uprights were constructed of built-up I-beam section, and have all a depth at the top of 0.82 ft., while the bottom depths are 1.64, 1.96, and 2.30 feet according to the lengths, the longest one being about 37 feet over all. The uprights were braced together in pairs 3.8 feet center to center with one hoisting chain to each pair, and these pairs were again fastened together in pairs,

four uprights being thus hoisted at once, using the two chains. This arrangement was adopted so that if the width of the curtains, about 7½ feet, should prove unmanageable, the latter could be cut in two and operated on the single pairs of uprights. However, no unusual trouble has been experienced with them, and the full width is still used. The top of each upright was hung in threaded suspender rods which could be raised by jack-screws, and would thus let the bottom of the upright swing clear of the sill in case ice or other troubles prevented its being hoisted by the chains. This provision, however, has been omitted in later dams, as the dangers anticipated have not been met with in practice, and in fact the apparatus was not used at Poses on one occasion when ice blocked the maneuvers (see p. 553).

The curtains (see Pl. 71) were made of creosoted yellow pine strips each 3 inches high and with one-sixteenth of an inch play between the strips to allow for swelling. The top strips were made 2 inches thick, and were strengthened with angle irons, and the bottom strips on the deep passes were made 3½ inches thick. The curtains were all made 7.47 feet in width, giving a clearance of 2 inches between neighboring ones, the longest curtain being about 17.55 feet over all. The bottoms were provided with heavy cast-iron shoes to assist in the unrolling, and the pins and hinges were made of bronze to prevent rusting. The curtains are not removed from the uprights except for repairs or cleaning.

Operation is done by electric winches provided with chain wheels at each end for the double chains, power being derived from the turbine and dynamo used for the maneuvers of the lock. The power available varies from 30 to 12 horse-power, the latter occurring when the head has been reduced by floods to about 3 feet. In case of breakdowns, a hand winch is available which can exert a total pull on the chains of 10,000 pounds.

The dam has proved very satisfactory It is usually kept up all winter, as the Seine rarely freezes. (See p. 552.)

Mirowitz Dam.—(See Fig. 225 and accompanying illustrations.)—This bridge dam is on the Moldau below Prague, and was completed in 1903 on a foundation entirely of river gravel. It is composed of a center pass of 184 feet clear span closed by gates and with 16½ feet on the sill, and two side weirs each with 10.2 feet on the sill and closed by trestles and needles, the one on the left bank being 65½ feet wide, and the other 180½ feet. A pass or chute for rafts is also provided on the right bank, 39.4 feet wide. The lift is 12 feet 9 inches, although during a season of very low water occurring shortly after construction the dam held for several weeks a head of 17 feet. The pass is closed by 25 uprights braced together in pairs in the usual way, and spaced 7.35 feet centers, the length of each upright being about 34 feet, and the distance from the sill to the bridge floor, 37 feet. No special means are provided, as was done at Poses, for removing the uprights

in case of trouble from ice or drift, as experience had not proved its necessity. The gates are about 6 feet wide and $17\frac{1}{2}$ feet high, reaching in one length from sill to pool, composed of iron buckle-plates $\frac{1}{4}$ inch thick, and moving between the uprights on Stoney rollers $3\frac{1}{2}$ inches in diameter and 3 inches long, spaced from 13 to 17 inches center to center. When hoisted they clear the upper pool level by about 2 feet. Each one is maneuvered by a single chain, as are also the uprights. The latter are built-up I-beam section, about $2\frac{1}{2}$ feet maximum depth, and tapered towards the top, where they are attached to the floorbeams by pins about 7 feet long. The clearance between each pair of frames is about $1\frac{3}{4}$ inches,

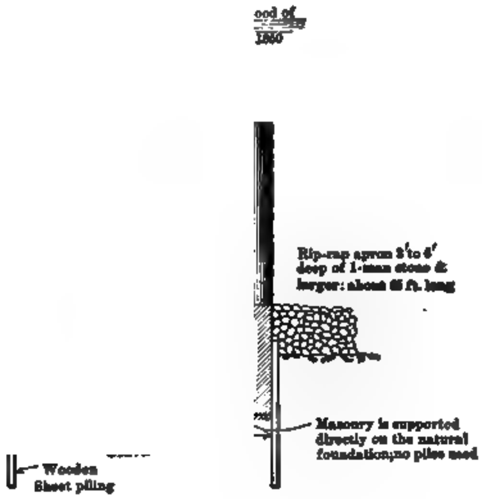


FIG. 225.—Section of the Pass of the Mirowitz Bridge Dam, Moldau River, Bohemia.

and their clearance above extreme high water when hoisted under the bridge, about 3 feet. A foot-walk is attached to the upstream side of the frames just above pool level. The bridge is built as a cantilever, and carries a highway with a solid floor, all the work being of very heavy construction, although considerably simpler than the dam at Poses.

Operation is done by electric winches, one being provided for the gates and one for the uprights, the power being derived from a vapor engine. The gate winch weighs about 3000 pounds and can exert a maximum pull of about 5000 pounds; the upright winch weighs about 5800 pounds and has a pull of about 8500 pounds. The time required for operation has been given on p. 557.

View from above Dam, Showing Dam in Position. Center Span or Pass closed by Gates, Side Spans or Weirs closed by Needles.

View Showing General Construction of Uprights, etc.

MIROWITZ BRIDGE DAM, BOHEMIA.

This dam has proved very successful, and the arrangement of the gates was found so convenient that all regulation is done by them, and the needle weirs are not handled except in floods.

Assiout Dam.*—This structure, the first of the great movable dams built on the Nile by the British, chiefly for the purposes of irrigation, was completed in 1902, and is typical of the later ones. The length from abutment to abutment is 2691 feet, divided by masonry piers into 111 bays each 16 feet 5 inches wide, the piers being 6 feet 7 inches wide except every ninth one, which was made 13 feet $1\frac{1}{2}$ inches wide for architectural effect. (See Pl. 73.) The discharge area provided is about 63,900 square feet. The tops of the piers are connected by arches, forming a continuous masonry bridge carrying a roadway 14 feet 9 inches wide, which serves also for operating purposes. The normal lift is 8 feet 2 inches, rising to 12 feet 3 inches in low water, and the normal depth on the sill is 15 feet 1 inch. The surface of the roadway is 41 feet above the sill. The openings are closed by steel sluice gates, each about 17 feet 5 inches long and 8 feet 3 inches high, moving in cast-iron grooves and hoisted by chains at each end operated by movable hand winches on the bridge. The lower gate is placed upstream of the other, so that the two can lie side by side when fully raised. Each gate is provided with four rollers about 10 inches diameter and 3 inches face, fastened to the ironwork. This avoids the complication of the Stoney roller, although it increases the friction. A lock $52\frac{1}{2}$ feet wide and $262\frac{1}{2}$ feet between gates was provided on the west bank.

The most interesting feature of this work was the foundation, as the natural bed consisted of fine sand, and was very unpromising as a foundation for any kind of dam. The details of the design are shown on Pl. 73. The cast-iron piling was made continuous from bank to bank, the joints being washed out and grouted after driving. The floor was made heavy enough to withstand upthrust, and an "inverted filter" of gravel was provided to catch any springs that might tend to carry out the particles of sand. Great trouble was experienced from springs during construction, nearly a thousand large ones being met with (see p. 381). The cofferdams consisted entirely of bags filled with river sand, a total of 2,697,000 being used, while the total yardage of the cofferdams was about 642,000 cubic yards. About 223,000 cubic yards of masonry were used on the entire work, including the lock, etc., the concrete in the foundation being a 1 to 3 to $6\frac{1}{2}$ mixture. The total cost including office work and all incidentals was about $4,500,000.

In 1903 a dam of similar design and purpose, and on a similar foundation, measuring 1168 feet between abutments, was completed in the delta of the Nile and is known as the "Zifta Barrage." This dam is noteworthy as carrying a

* See Proceedings Inst. C. E., vol. clviii, 1904, paper by G. H. Stephens, through whose courtesy the cuts are reproduced.

depth of water on the floor of 19 feet 8½ inches. Other dams have since been projected in continuation of the system of storage and irrigation, and one of them has been completed at Esneh.

Dams on the Oise.—This river has some bridge-and-gate dams of interesting design, made with the object of being kept in place all winter so that traffic could be maintained even in severe weather. The first two, at Creil and Ile-Adam, were finished in 1901, and are similar in general features. The Creil dam (see Pl. 74) is built on a foundation consisting largely of sand, and is provided with two passes each 98½ feet wide, a sluice 43.3 feet wide, and a lock 39 feet wide. The pass sills are 10⅓ feet below the upper pool, and the sluice sill 13 feet, the lift being 4¾ feet. The arrangements are as follows: From a fixed upper bridge, which gives a head-room of about 13 feet above the highest navigable water, are suspended 3 light counterweighted footbridges about 8 feet wide (one for each opening), which can be hoisted up under the floorbeams of the upper bridges or lowered till the footway is about 3 feet above the upper pool. To the upstream side of the footbridges can be attached the removable I-beam uprights, spaced about 4 feet 1½ inches (1.25 meters) centers and about 13 feet long, which support the Boulé gates. The latter on the passes are of wood in 7 ranks, each gate being about 4 feet long, 16½ inches high, and 1⅝ inches thick, except the top rank, which is about 10 inches high. On the sluice the uprights also are of I-beams, but 14 feet 5 inches centers, and carry iron gates 19½ inches high and about 14 feet 4 inches long. Each pass gate except the top ones is provided with six of the ball rollers described on p. 613, and each sluice gate has four. The pass gates are maneuvered entirely by hand (using a hook-pole), and without special difficulty under a 4¾-foot head, while the sluice gates are maneuvered by a hand winch on the fixed bridge, the ends of the gates being provided with removable hoisting rods whose tops are attached to the footbridge, so that the gates can always be reached for maneuvering. When the dam is to be opened the gates and uprights of the passes are taken up and carried ashore on hand trucks, and the sluice gates and uprights are placed on a boat, on account of the greater trouble in handling them. The footbridges are then hoisted by hand winches and the river is left clear.

This dam has given excellent service (except for some trouble with undermining from above, as described on p. 374) and has been kept in place and maneuvered successfully during severe winters.

Mohawk River Dams.*—In 1906 the construction was commenced of a series of eight bridge dams on the Mohawk River between Schenectady and St. Johnsville in order to provide canalization for the New York State Barge Canal. (See also pp. 545 and 623.) With the exception of the two upper ones all the bridges were designed to carry highways. The area of opening given by these dams was made

* See also Tables of Locks and Dams at the end of the book.

Dam at Crane's Village during Erection of Steel Work. This dam has one center span of 180 feet clear and two side spans each of 150 feet clear. Depth on sill, 20 feet; normal lift, 15 feet; uprights, 15 foot centers; gates 29 feet 8 inches long, in two tiers. All masonry is of concrete.

VIEW OF BRIDGE DAM, MOHAWK RIVER, N. Y. (NEW YORK STATE BARGE CANAL. See Pl. 72.)

Part of the dam at Fort Plain, showing maneuvers of the gates and uprights. The pair of uprights on the left are being lowered into place; the pair on the right have been lowered; and on the center pair the large gate is resting on the masonry sill and the small gate has been partly lowered.

Dam at Crane's Village in operation, sustaining a low-water head of 17 feet.

VIEWS OF BRIDGE DAMS, MOHAWK RIVER, N. Y (NEW YORK STATE BARGE CANAL).

Placing a Dam in Operation. The gates of the lower tier are resting on the masonry sill, with the exception of the three shown, which are about to be lowered the final 2 feet. The upper gates have not yet been lowered.

Regulation by Overflow and by Maneuvers of Gates, showing one of the upper tiers of gates raised to assist in passing a small flood.

VIEWS OF BRIDGE DAMS, MOHAWK RIVER, N. Y. (NEW YORK STATE BARGE CANAL.)

practically the same as that of the natural river bed at each site, and from two to three spans were used according to the width of the river, the sill masonry being placed at the same level for each dam all the way across. The depth on the sills was made from 16 to 20 feet, and the lifts vary from 8 to 15 feet. The clear widths between piers varies from 150 to 210 feet, except that in one case, owing to the position of the bedrock and to the risk incurred from the annual ice gorges at that point, two spans of 240 feet each were used. The variation of length due to temperature, which would swing the uprights out of the vertical plane as the bridge expanded or contracted, was provided for by using several vertical plates for hangers for the uprights instead of a single thick one. The aggregate length of net opening of the dams is 3750 feet; the total length including piers is 3862 feet.

In order to reduce to a minimum the number of pieces to be handled, and thus simplify the operation and maintenance, as well as to reduce the cost of construction, wide spacings and large sizes were used throughout. Thus the gates in all the dams were made 29 feet 8 inches in length and only two rows were used, the lower row, where the depth on the sill was 20 feet, being about 13 feet in height the upper row about 7 feet. In addition to these, two small gates about 2 feet in height were provided at each dam for passing débris, etc. The gates are built of I-beams and buckle-plates and are mounted on rollers fixed to their downstream sides. These move in channel irons placed on the upstream faces of the uprights, which are spaced 15 feet centers and lightly braced together in pairs, the gates therefore overhanging nearly $7\frac{1}{2}$ feet at each end. The spaces between the lower gates are closed by hinged iron cover-plates, and those between the upper ones by wooden strips. The gates rest on the uprights when hoisted. (Pl. 72.) The maximum length of upright was about 44 feet 6 inches.

Operation is provided for by two chains on each gate and on each pair of uprights, spaced about 15 feet apart and designed for use with steam winches provided with chain wheels at each end of the shaft, so that each pair of chains moves as a unit. Two duplicate winches are provided for each dam, of about 40 horse-power each, and each winch has three different sizes of chain-wheels, so it can operate any of the parts of the dam. The lifting capacity is sufficient to permit both rows of gates to be raised under a full head of water by hoisting the lower row, in case the chains of the upper row are injured. The upstream and downstream tracks connect, so that one winch can move to any part of the bridge.

All the masonry is of concrete, supported in most cases on piles, and with cut-off walls and lines of sheet piling from bank to bank. The foundations varied from a sandy clay to limestone rock, several of the dams being built on gravel. The locks were made 45 feet in width and 300 to 310 feet in available length, with a least depth of 12 feet over the miter sills.

At times when the dams are out of position the masonry foundations act

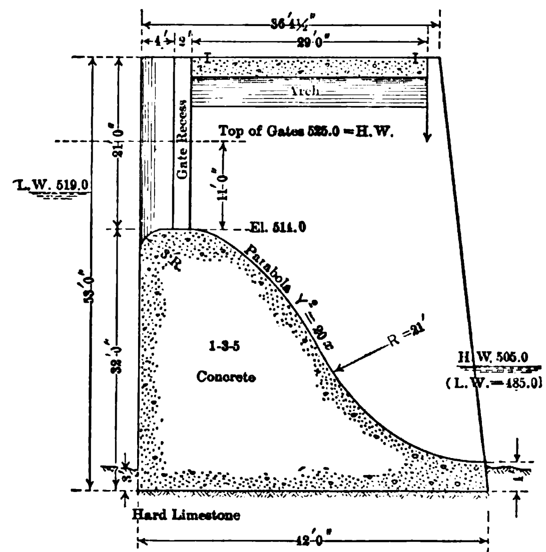
as weirs in passing the discharge. This is due to the fact that at the upper end of each pool extensive dredging was required in order to secure the 12-ft. depth of channel, the slope of the river being very steep. The natural low-water level below each dam was thus lowered several feet, while just above the dam, where the river bed was left in its natural state, it was kept unchanged. In several cases it was necessary to extend the upstream riprap protection from 20 to 30 feet above the masonry, sloping it gradually upward until its edge about coincided with the natural low-water level and kept it at its original height. Without this precaution the flow in approaching the dam when the latter was not in place would have passed over a steep unprotected slope, and would have begun to scour out the river-bed and would have continued this process until the original slope had been restored, approximately, in a manner similar to that described on p. 68. Although the slope of the bed for some distance below the dams was entirely changed by the dredging, it was not anticipated that scour would result in those portions, as the bottom of the channel was made level, thus giving a larger area of discharge than the natural one and reducing the velocity accordingly.

The first dam was put in commission in 1911 and others followed in 1912. They proved to be very satisfactory in operation.

Keokuk Dam.*—A large dam of the masonry bridge type is to be found at Keokuk, Iowa, on the Mississippi, having been practically completed in 1912. It was built to utilize the water-power of the rapids at that point. The dam proper is 4278 feet long between abutments and contains 119 openings of 30 feet clear each, separated by piers 6 feet in thickness. These openings have each a spillway 32 feet high, 42 feet wide at the base, with a vertical upstream face and an ogee curve for the downstream face, and above the spillway is a single steel gate 11 feet high and 32 feet wide sliding vertically in grooves in the piers. (Fig. 226.) The 119 gates will be operated by a traveling crane running on rails on the top of the structure. The maximum depth of water over the spillway will be 11 feet, and the maximum depth of water at the upper side of the dam will be about 40 feet, the steel gates controlling the flow in different stages of the river. The lock, situated below and west of the power house, is 110 feet wide and 400 feet in interior length, with a lift of 40 feet and a minimum depth of 8 feet on the miter sills. There is from 200,000 to 300,000 horse-power capable of development. The dam provides a minimum depth of about 6 feet in the channel for 65 miles of river where formerly the rapids had to be passed in a canal with three locks. All the structures are of monolithic concrete, with footings in hard, blue limestone, and the entire work, including the dam, power house, lock, dry dock and protection wall has a length of 9096 feet. The concrete was made a 1 : 3 : 5 mixture.

* For a general description see "Engineering News," September 28, 1911.

St. Andrew's Rapids Dam.—A large bridge dam of the Caméré type is to be found at the St. Andrew's Rapids of the Red River in Manitoba, about 20 miles below the city of Winnipeg, its construction having been completed about 1911. It provides access between that city and Lake Winnipeg. There are six spans in the dam, each truss being 126 feet 8 inches long and 21 feet deep between chords, and each span being composed of three trusses. A seventh span carries the roadway across the lock. The clear openings between piers are about 120 feet each. The piers are 14 feet thick, 55 feet long at the top and 76 feet long at the base, their average height being 50 feet. The bridge carries a highway between the top chords, and the operating machinery, etc., is carried on tracks between the bottom chords. Between the piers is a concrete sill or substructure on which rest the movable



GENERAL SECTION OF KEOKUK DAM

FIG. 226.

portions of the dam; this sill measures about 38 feet in width (parallel to the stream) and 20 feet in maximum height, and has its top about 7½ feet above extreme low water. The upper pool level is about 13½ feet above the top of the sill, thus making a total low-water lift of about 21 feet (Pl. 75).

The frames or uprights carrying the curtains are 34 feet 8 inches long, measured from the hinge, and 2 feet 3 inches deep, tapering in the upper third. They are rigidly braced together so as to make sets of two and fours, placed alternately, two chains being used to hoist each set. Each curtain covers the full width of one set, with the necessary overlap; thus where one curtain covers two frames, the adjoining curtain covers four frames; the next, two frames, and so on. The curtains are all 13 feet 7½ inches in height and 7 feet 7¾ inches in width, the laths being joined by copper links and fastenings with phosphor-bronze pins. These

laths are all of long-leaf yellow pine and are 3 inches in face width, varying from $1\frac{21}{32}$ inch in thickness for the top ones to $3\frac{5}{32}$ inches for the bottom ones. The curtains are operated by a single chain. Each of the six spans has 46 frames and 15 curtains, making 90 curtains in all and 20 extra ones were provided for contingencies.

For hoisting the large frames (consisting of four girders) two 20-horse-power electric winches are provided, and for the small 2-girder frames there are two similar winches, but of 10 horse-power each. For the curtains there are three 2-horse-power electric winches, all alike, and capable of being also operated by hand. All the chains are engaged over chain wheels.

The lock is 45 feet wide and 215 feet between hollow quoins, composed, as is the masonry of the dam, of $1 : 2\frac{1}{2} : 5$ concrete. All the structures rest on rock.*

COST OF DAMS WITH GATES OR CURTAINS

Location.	Lift, Feet.	Depths on Sill, Feet.	Cost per Linear Foot.		
			Fixed Parts. $	Movable Parts. $	Total. $
Suresnes, just below Paris (1885)[1]	10.7	10.0, 13.3 and 15.0	....	15.00 [2] 34.00 [3]	632.00 [4] 866.00 [5]
Poses, Lower Seine (1885)[6]	13.7	9.8 and 16.4	814.00 for masonry; 114.50 for bridges (average)	53.50 for uprights; 26.00 for curtains (average)	1008.00 [7] 1716.00 [8]
Mirowitz, Moldau (1903)[9]	12.75	16.5	....	....	[10]
Assiout, Nile[11]	8.2	15.2	....	....	1550.00 [12] 1640.00 [13]
Mohawk River, N. Y., Dam No. 8, Tribes Hill	11.0	16.0	228.00	72.00 [14] 108.00 [15]	408.00 [16]
Mohawk River, N. Y , Dam No. 5, Rotterdam Junction	15.0	20.0	328.00	88.00 [14] 113.00[15]	529.00 [16]

[1] Trestle dam with gates and curtains. See description on p. 618 and after.
[2] Cost for gates only of navigable pass, and not including trestles, etc.
[3] Cost for curtains only of same.
[4] Average cost of whole dam without protective work, etc.
[5] Average cost with protective work, dwellings, etc. Total cost of dam, all included, $560,000. Cost of lock, etc., was additional.
[6] Bridge dam with curtains. No highway. See description on p. 626 and after.
[7] Average cost without protective work, etc.
[8] Average cost with protective work and all incidentals.
[9] Bridge dam with a gate pass and two needle weirs. See description on p. 628 and after.
[10] Average cost with all incidentals is stated to have been from $2000 to $3000 per foot run. Bridge carries highway and has a solid floor.
[11] Bridge and gate dam. See description on p. 631.
[12] Cost of construction only; includes a lock and regulating works, and is based on a length of 2691 feet for the dam itself.
[13] Cost of all works, including engineering, etc. Grand total, about $4,500,000.
[14] Cost of gates, uprights, etc.
[15] Cost of bridges proper. Dam 5 was all on piles, and had a wider apron, etc., than Dam 8, half of which was on rock.
[16] This includes all expenses for piers, abutments, bank protection, ironwork, etc. See p. 632 and after for general description of these dams.

* "Engineering Record," March 26, 1910.

SHUTTER DAMS.

General.—The development of the shutter dam is due to M. Thénard, who conceived the idea of the type from certain weirs on the river Orb, in France, which had been provided with small movable shutters during the eighteenth century.

The Thénard shutter consists of a panel hinged to a floor at the bottom and supported by a prop near its middle on the lower side, the shutter lying down behind a sill when lowered. To facilitate the lowering M. Thénard introduced a bar provided with projections, and capable of being pulled or pushed by means of gearing in the masonry. It was arranged so that the projections would strike the props one at a time and pull them from their supports, thus allowing the shutters to fall. This was known as the tripping-bar. (See p. 589.) It was found difficult to raise them, however, against head, and for that reason counter-shutters were introduced upstream. These were arranged to rise downstream, or in the opposite direction to the main shutters, being held in an upright position by chains (Fig. 226*a*).

Maneuvers.—To raise the dam, the counter-shutters, which were kept in place on the floor by latches worked by a tripping-rod, were released, and the force of the current caught them and swung them up and over until they were stopped by their chains in a position nearly vertical. This dammed up the water, and the workmen then raised the main shutters by hand. The space between the two rows was then filled with water by opening curtain-valves, and the counter-shutters were pushed down and latched. The lowering was done with the tripping-bar. When raising the shutters after a flood the workmen sometimes had difficulty in finishing the maneuvers before the water rose over the counter-shutters.

Examples.—Four dams of this type were built in France, the last one, that of St. Antoine, having been completed in 1843. The shutters in this dam were 5.6 feet high and 3.9 feet wide. In the others they were 3.3 feet high and 6.6 feet wide, the length of the weirs being 156 feet and that of the passes 74 feet.

A dam with a pass closed by Thénard shutters was also built in 1850 at Courbeton on the Seine, but with needles and trestles substituted for the counter-shutters. The weir was closed by needles alone (Fig. 226*a*). The pass was 39.7 feet long and the shutters were lowered by a tripping-bar, worked by a turbine. The action of the latter was automatic, starting or stopping as the pool level rose or fell.

Other examples are to be found in India, on the Mahanuddee and Cossye rivers (Figs. 227 and 228), and on the Sone Canal, where they are used to close flushing-sluices. Some of them are over 21 feet in length and support heads of nearly 10 feet. Much trouble was experienced with the breaking of the chains of the counter-shutters, due to the sudden jerk when the latter came to place. This was finally overcome successfully by using a hydraulic brake on the upstream side, consisting of

a piston working in a cylinder pierced with small holes, the piston being attached to the shutter, and the cylinder to the floor. As the shutter rises it draws the piston through the cylinder, and the water escapes slowly through the holes, thus checking the rapidity of its movement.

The shutters of the Mahanuddee weir are about 7 feet wide and from $7\frac{1}{2}$ feet to 9 feet high. There are ten bays or openings in the dam, each 50 feet wide

FIG. 226a.—Details of Thénard Dams.

and separated by piers 5 feet in width in which the gearing for working the shutters is placed. Each bay contains 7 shutters.

Girard Shutter.—To overcome the difficulty of raising the Thénard shutter M. Girard proposed to do away with the counter-shutters and to substitute for the props hydraulic jacks, which would raise or lower the main shutters at will. Seven of this type were constructed in 1870 at Auxerre on the Yonne, and gave excellent

satisfaction. The shutters were 11½ feet wide and 6½ feet high, the cylinder of the jack being 12 inches in diameter. On the introduction or withdrawal of the power the piston-rod moved out or in, thus raising or lowering the shutter. (Fig. 229.)

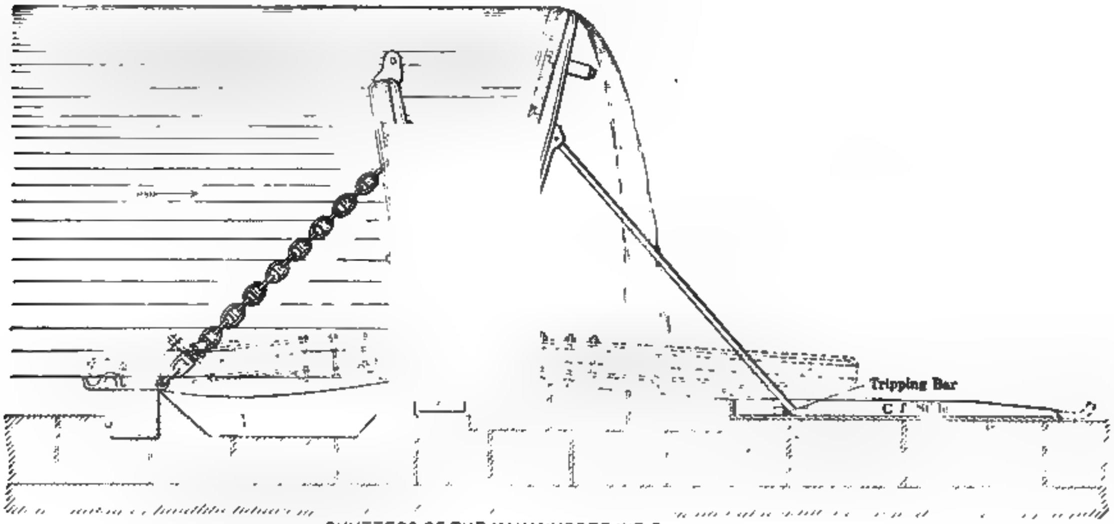


FIG. 227.

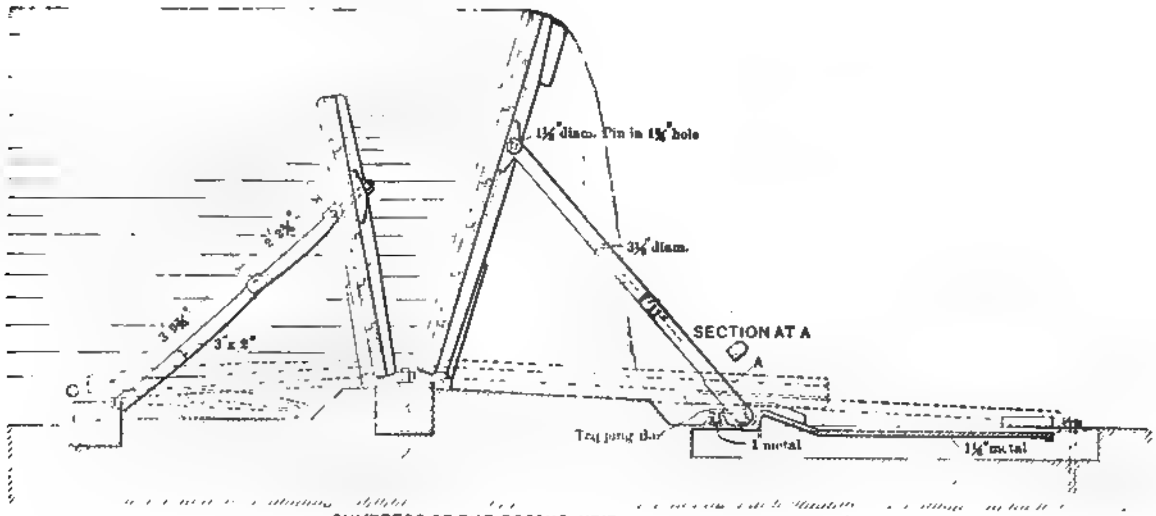


FIG. 228.

The cost of the dam per lineal foot was $60 for the fixed parts and $119 for the movable parts, or a total of $179.

Owing to the untimely death of its inventor during the Franco-Prussian war no further examples of this type were constructed.

Remarks.—The shutter dam possesses the advantages of simplicity, and of not being easily injured by drift or ice, since there are few parts in which these can

FIG. 229.—Section of a Girard Shutter.

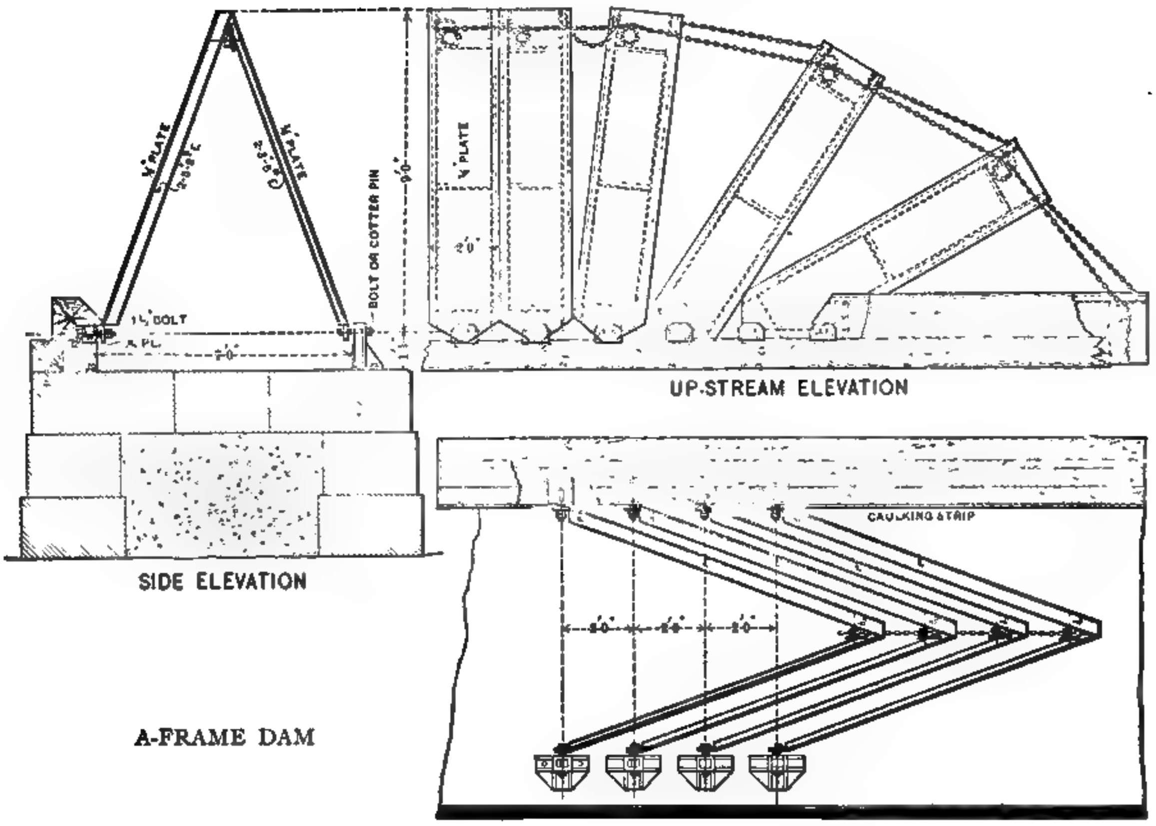


FIG. 229a.

GENERAL VIEW OF DAM No. 6, OHIO RIVER

Showing the lock and Chanoine wicket pass in the distance; one bear-trap weir rising into position and the other at full height; and the A-frame weir in the foreground, with part of the cofferdam still in place.

be caught. The difficulty of raising it, however, and the expense of the counter-shutters, proved a serious drawback, and after the introduction of the Chanoine wicket the type became practically obsolete.

A-frame Dam.—This dam is composed of a series of adjoining trestles provided with two legs or main members only, these being inclined at such an angle that when the dam is raised the upstream ones are in contact, and when lowered the trestles or frames "nest" or lie down one inside the other. The upstream members are intended to have a width of about 2 feet, and this wide face holds back the water, the type thus combining in one the water-curtain and its support. (See Fig. 229*a* and Pl. 76, and accompanying cut of Dam No. 6, Ohio River.) The trestles can be operated by the customary methods of chains and latches.

An experimental weir, 120 feet in length and with 13 feet 2 inches on the sill, was constructed at Dam No. 6 on the Ohio River in 1899. The location, however, and the general conditions were unfavorable, as the structure was placed on the convex bank and with its sill considerably below the natural bed of the river, and the artificial depression thus formed filled to the original level during the first winter's floods, burying the trestles under several feet of gravel. They were raised later, but it was deemed unwise to lower them with the certainty of the gravel covering them again, and the dam has in consequence acted since that time as a stationary structure, its merits or demerits being unknown.

CHAPTER IX.

DRUM WICKETS, BEAR-TRAPS, AND ROLLING DAMS.

DRUM WICKETS.

The drum wicket was invented by M. Desfontaines, and applied first in 1857 at Daméry, on the Marne, a tributary of the Seine. After its success was assured, others were built on the same river, the best known being that of Joinville, finished in 1867. Other examples are to be found in Germany, generally as sluices, their lengths varying from 17 to 39½ feet, with depths of water on sills from 5.6 to 9.2

Fig. 230.—Drum Weir on the Spree, Germany.

feet, the one at Charlottenburg on the Spree just below Berlin (built in 1885), consisting of a single wicket 32.8 feet long and 18½ feet high with a normal head of 3½ feet (Fig. 230). One type has been used in America of a modified design known as the Chittenden drum (Fig. 232). A lock-gate of this design was built for lock No. 2 of the Mississippi River, near St. Paul, and a few fixed dams have been provided with movable crests of this type. (See pp. 521 and 649, and Pl. 54.)

Description.—The Desfontaines wicket (Fig. 231) consists of two diaphragms or leaves in approximately the same plane, one above and one below, joined together and turning on a horizontal axis. The upper one is exposed; the lower one works in a closed chamber which is filled or emptied by culverts closed with valves. To raise the dam the water from the upper pool, which must of course have more or less head, is admitted to the chamber, and by pressing on the lower leaf causes the whole wicket to revolve, thus raising the upper leaf. To lower the dam, the valve connecting with the upper pool is closed, and that connecting with the lower pool is opened, allowing the water in the chamber to escape, when the upper pool

pushes over the upper leaf and forces it down behind its sill. The operating valves are placed in the abutments or in piers in the dam. The lower arm is usually made of the same length as the upper one, or a little longer.

Owing to the necessity for the chamber, which requires a considerable depth of foundation, and to the need of a head of water for operation, these wickets have only been used for weirs.

Drum Wickets on the Marne.—A description of the weirs on this river will illustrate the general features of the type. There are twelve dams between Epernay and Charenton provided with weirs of drum wickets, the length of opening covered by them varying from $98\frac{1}{2}$ to $206\frac{1}{2}$ feet. The first one was completed at Daméry in 1857, and the last one at Noisiel in 1887, forming the only system of importance

FIG. 231.—Section of Desfontaines Drum Dam, River Marne. (1867.)

in which a series of this type is to be found, except the five needle dams on the Main between Frankfort and Mayence, on each of which a drum sluice was provided $39\frac{1}{2}$ feet long with 5.6 feet on the sill.

At the dam at Joinville on the Marne, which is typical of the rest in general features, the pass is 39.4 feet long, closed by needles, and the weir is $206\frac{1}{2}$ feet long, closed by drum wickets. The lift is 7.1 feet. Each wicket is about 5 feet long (1.5 m.) and 3.3 feet high above the sill, and independent of its neighbors, from which it is separated by cast-iron diaphragms provided with openings to permit the water to circulate freely along all the wickets. (See Fig. 231.) The recess or chamber is covered on the upstream side by a horizontal plate of wrought iron, and on the downstream side by a plate of cast iron. The wickets are 42 in number and are made of $\frac{3}{16}$-inch (5 mm.) wrought-iron plate, and the downstream edges of each

lower leaf are provided with rubber strips which fit against the masonry or metalwork when the dam is raised and thus secure a water-tight joint. The upper and lower leaves of the wicket are of almost equal length, but it was found after completion that it would have been better to have made the lower leaves the longer of the two, as owing to the small size of the culverts, to the friction of the parts, and to the dynamic pressure of the overflow, a head of 24 to 30 inches is needed to make the wickets rise easily against a current.

To raise the dam, the culvert is connected with the upper pool by means of a valve, and the water enters the chamber and passes through the upstream openings in the diaphragms, pushing up one wicket after another till all are upright. The water on the downstream side escapes through the lower openings. To lower the dam, the first valve is closed and another one opened connecting with the lower pool; the head from the upper pool acting on the upper leaves then forces down the wickets slowly, and they in turn push back the water in the chamber (on the upstream side) through the openings by which it entered, while the water from the lower pool comes in through the downstream openings and fills the displacement. One set of valves is provided in the pier and one in the abutment, and if a full head is available (7.1 feet) the entire weir can be raised or lowered in a few minutes. A reduced head involves a corresponding increase of time.

As a close regulation of the pool would need constant attention, and as the arrangements do not permit the raising or lowering of a few wickets only (the chamber openings being uninterrupted from end to end of the dam), M. Desfontaines introduced on nine of the dams a series of props fastened to the downstream side of each upper leaf and supporting it at an angle of 30° to 45°, on the same principle as the prop of a Chanoine or Thénard wicket. The lower ends of the props when supporting the wickets rest against a species of tripping-bar. If a rise comes which requires a partial lowering of the dam, the corresponding valves are opened, the wickets revolve, and the props slide back until they rest against supports on the tripping-bar which prevent them going further. If the dam has to be lowered entirely, the bar is moved by gearing on the pier or abutment, the supports are pulled away, and the props and wickets lie down. This artificial support was found necessary for another reason; when the lower pool rose about a foot above the hinge and there was more or less discharge over the crest, the impact and disturbance of the water forced down the wickets, and they would not rise until the pool downstream had fallen to a level below the hinge.

Many years after completion all the dams were raised to give an additional depth of 20 inches. This was done by placing loose planks or flashboards on the top of each wicket, either removing them by a boat if the dam had to be lowered, or letting the water carry them off and picking them up below. The method has been found very satisfactory.

Drum Wickets in America.*—A modification of the Desfontaines type was proposed by Colonel H. M. Chittenden, Corps of Engineers, U. S. A., and several examples of it have been built in the United States. It consists (see Fig. 232) of a sector of a circle, air tight, and working in a closed chamber. The water is admitted under head into this chamber and pushes up the wicket; when the head is shut off and the chamber put into communication with the lower pool, the wicket is pushed down into the recess. The maneuvers in short are carried out on the same principle as in the original type. A weir of this design was completed on the Osage River, Missouri, in 1911. The pass in this dam is 415 feet long and is of Chanoine wickets. The drum part, or weir, is 375 feet long and has five openings of 75 feet each, closed by Chittenden drums. The top of the masonry is about

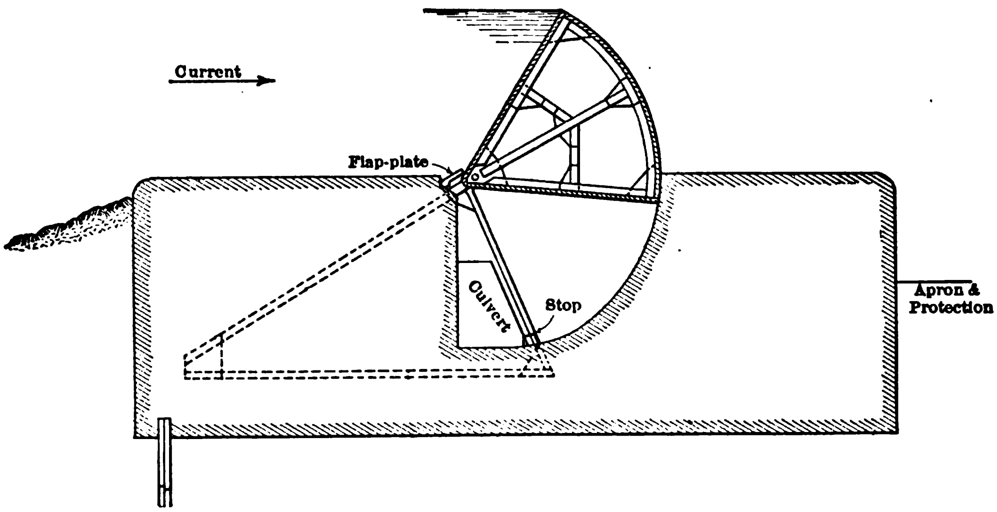


FIG. 232.—Typical Section of a Chittenden Drum Wicket.

9 feet above the river-bed and the depth on the sill is 7 feet. The drums are separated by piers 10 feet in thickness. They have been found to fill with mud, and require frequent partial raising by mechanical power so as to permit cleaning. When thus raised about 2 feet, and cleaned, they operate properly to full height.

This design, as compared with the Desfontaines wicket, has the advantages of simpler construction and freedom from liability to drift getting wedged under the leaves. On the other hand the weight is unbalanced, and as the drum is a closed vessel, it will either float up during floods if of light specific gravity, or if heavy will require a considerable head to push it up when it has to be raised. At Dam No. 2 on the Monongahela close to Pittsburg (see p. 522 and Pl. 54) a movable crest of these wickets, 3 feet in height, was constructed in 1905 of steel plates, and the feature of unbalanced weight was controlled by an air compressor on shore forcing air into the drums through special joints in the hinges. Holes 1 inch in diameter were

* See Pl. 46 for locations of rivers mentioned.

left every few feet in the bottom corners of the drums through which the water escaped when air was being forced in, and in floods the same holes readmitted the water and the air flowed back through the pipes, removing all tendency to flotation. This dam is divided by a central pier into two lengths, one of 390 feet and one of 400 feet, each section being provided with a continuous line of separate drum wickets, and each wicket being about 40 feet long. The valves and pipes are arranged so that half of each section is worked from the nearest masonry, thus making four independent divisions. The wickets have operated in general satisfactorily, and under an ordinary head will all rise or fall in a minute or two. Experience has shown, however, that drift and sediment may produce unlooked-for complications, as at one time some of the flap-plates (see Fig. 232) were torn off, and at another, some of the wickets after a long immersion were found to contain a sufficient amount of sediment inside to prevent their operation. For this and other reasons manholes should always be provided for access to the interior. A movable crest of similar design is to be found at Dam No. 3 on the same river.

If conditions were such that backwater did not submerge any of the drums (as for example on a dam of very high lift) no air-compressor would be needed. By leaving holes as described for the water to escape, and providing communication with the outer air through the hinge, the complications of unbalanced weight would be largely removed.

Care must be taken in sediment-bearing streams to prevent the narrow space between the curved faces of the wicket and its chamber from becoming filled with sand when the wicket is down, as it has been found that if this happens the friction will prevent the rising.

At the Joliet power-house of the Chicago Drainage Canal are two steel drum wickets, one 48 feet and one 12 feet long, each with 19 feet on the sill, and used to regulate the water surface of the canal pool. The wickets are shaped somewhat like the Chittenden wicket (Fig. 232), but are reversed, as the water is supported by the curved face, and the action on the hinge is therefore a thrust instead of a pull. The curved and the upper sides only are plated, the water pressure thus acting against the upper side and holding up the wicket. This side is 26 feet long, but experience has shown that a length of about 36 feet would have been better, as with a depth of 9 feet or more flowing over the top the wicket is sensitive. The full head of 19 feet is available for operation, and the two wickets together can pass 5000 cubic feet per second. The design has given excellent satisfaction.*

Remarks.—The drum wicket is in some ways the simplest of movable dams and is easy and certain in operation, and had its special features been better known it would doubtless have been more widely applied. As a crest for weirs, where it is desired to have a quick and easy means of regulating the water level, it offers

* For description see Engineering News, November 12, 1908.

many advantages. On the other hand it has the disadvantages of all so-called automatic dams, namely, that if it gets out of order it is difficult to examine and repair. The weirs of the Marne, however, where the river is of moderate regimen and the amount of sediment small, appear to have given every satisfaction. Provision should be made when building so that temporary cofferdams can be placed to maintain the pool level in case the drums get out of order. It is necessary to provide culverts of ample size to allow for leakage, etc., and to remember in proportioning the parts that the dynamic effect of the water, when the wicket is to be raised against a current, will mean a loss of head of from 1 to 2 feet, as described for the dams on the Marne.

Cost of Drum Dams.—The cost of the Joinville weir per lineal foot is given as $105 for the fixed parts and $48 for the movable parts, or a total of $153.

BEAR-TRAP DAMS.

General.—The bear-trap was the pioneer of movable dams, although it was practically unknown until within recent years. The first one was built by White & Hazard in 1818 on the Lehigh River in the United States, and in the year

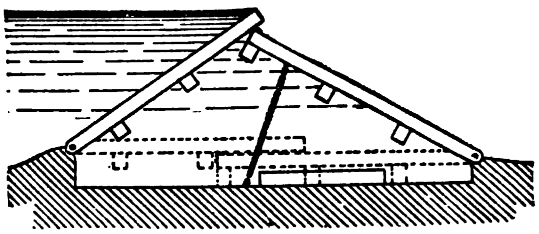


FIG. 233.—Section of Old Bear-trap Dam.

following twelve more were built. The type disappeared, however, for forty or fifty years, except for occasional examples on lumbering streams, the next one applied to navigation being that on the river Marne in France, where a gate was built 28 feet 8 inches long and $9\frac{1}{2}$ feet high above the sill. It was so badly proportioned that it proved a failure, after having cost $488 per lineal foot to construct, and as a result the type was generally condemned. In the last few years, however, other examples have been built in America of lengths from 14 to 120 feet and of heights from 7 to 14 feet, and have shown that with proper designing the bear-trap is a valuable device.

Description.—The bear-trap, as usually built, consists of three varieties, the original or "old" type, the Parker, and the Lang. The first (Fig. 233) consists of two straight leaves, hinged at the bottom, the upstream leaf overlapping the downstream one when lowered, and when water is introduced underneath both leaves are pushed up, the end of the downstream one sliding along and helping to push up the other.

In the Parker type (Fig. 234) the upstream leaf is divided into two parts, hinged together, so as to save width in the foundation, and the tops of both leaves are hinged together, thus avoiding the sliding friction of the old type. This is known as the Parker bear-trap and also as the "direct Parker." Sometimes the downstream leaf is divided instead of the upstream one, in which case the structure is known as the reversed Parker (Fig. 235). The Parker type was independently suggested early in the history of movable dams by M. Girard, a French engineer.

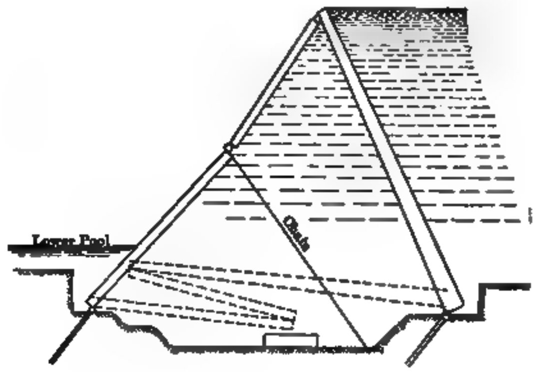


FIG. 234.—Section of Direct Parker Bear-trap Dam with Idler Leaf.

FIG. 235.—Section of a Reversed Parker Bear-trap Dam.

The Lang gate (Fig. 236) is the same as the Parker, except that the upper part of the upstream leaf is replaced by a chain, and the opening covered by a sliding "idler" leaf.

Many other varieties have been proposed, but, with the exception of one built for a regulating-weir on the Chicago Drainage Canal (Fig. 238), none has come into use.

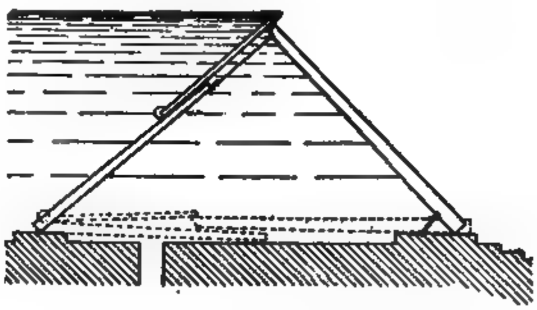


FIG. 236.—Section of Lang Bear-trap Dam.

Bear-trap Dams in America.—America is practically the only country where this dam has been employed, and examples of all three types have been tried. The "old" bear-trap has been in use for many years on the weirs of the movable dams of the Ohio and Allegheny rivers, in lengths up to 120 feet and with a maximum depth 13 feet 2 inches of water in the sill. A wooden gate of the Parker

type was built on the Muscle Shoals Canal in Tennessee in 1892, with a length of 40 feet and a vertical height of 8½ feet, and another gate of steel on the Louisville and Portland Canal in 1896, with a length of 40 feet and a height of 15 feet 3½ inches (Fig. 237). Three examples of the Lang Gate were built at the Sandy Lake Reservoir (at the headwaters of the Mississippi in Minnesota) in 1895, the two largest being each 40 feet long and 13 feet high, made of wood. In addition to these, several Parker and Lang bear-traps were built by lumbermen and others in the Northwest.

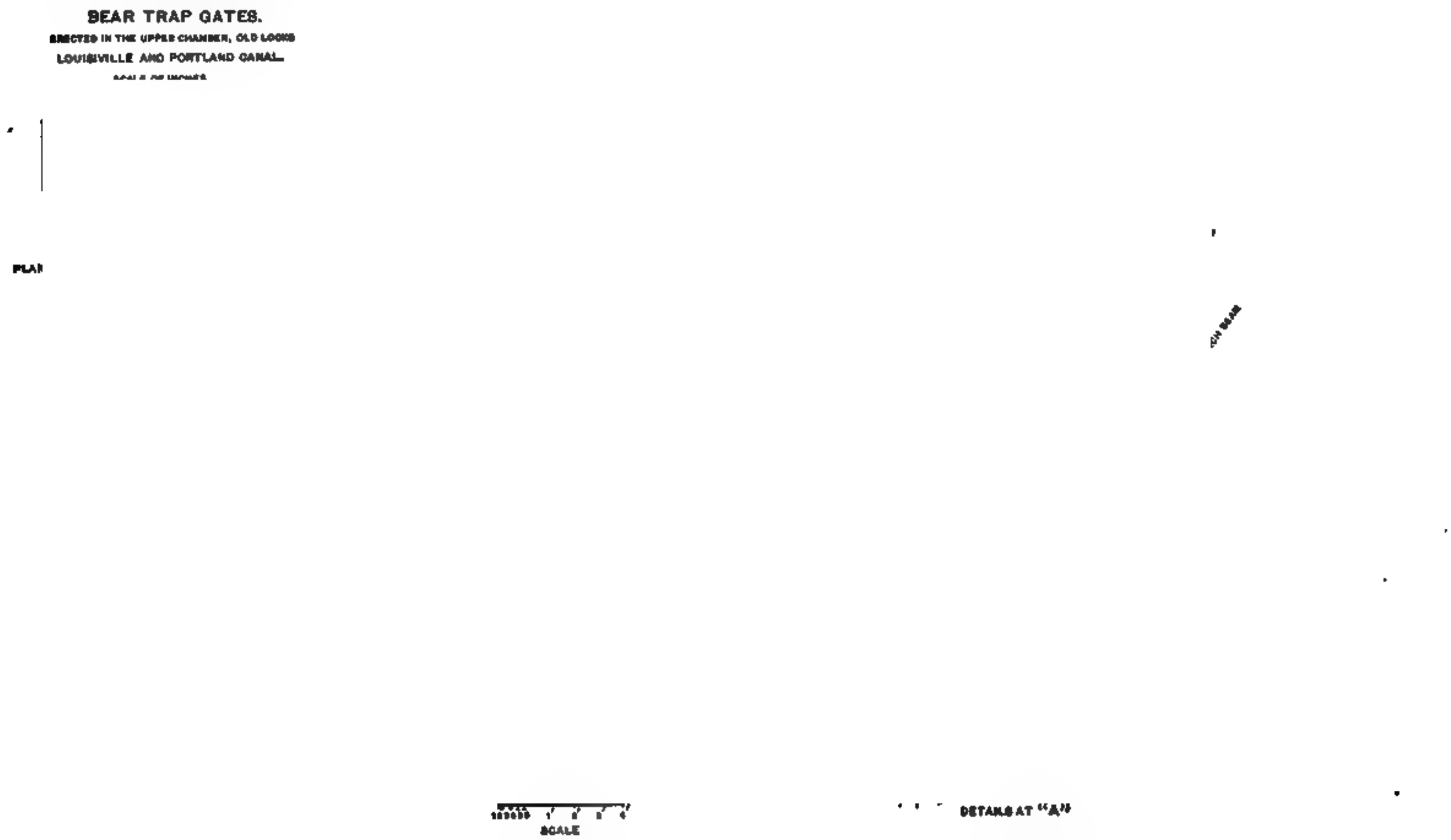


FIG. 237.—Bear-trap at Louisville, Ky., 1896. (Direct Parker Type. Depth about 15 Feet on the Sill.)

The bear-trap on the Chicago Drainage Canal (Fig. 238), completed about 1898, is used for flushing and regulating purposes, and spans a clear opening of 160 feet with a vertical movement of about 17 feet, making it the largest bear-trap in existence. It is of steel, and to guard against warping a system of counterweights and hydraulic jacks is provided at each end. The design was considerably more elaborate than any one of the three standard types, but it is stated that the gate has worked well and has been kept in operation winter and summer.*

* For a description and drawings see Engineering News, March 24, and May 26, 1898.

The first bear-trap on the Ohio was built at the Davis Island dam just below Pittsburg in 1889, and was of wood and 52 feet in length, and considering the lack of experience with the type, gave excellent satisfaction. The base width was 33 feet 10 inches center to center of hinges, while the downstream leaf was about 23 feet long and the upstream leaf 16 feet 6 inches long, the distance from the upstream pin to the pool level being 9 feet $8\frac{1}{2}$ inches. (See Fig. 241, p. 659.) The base was somewhat too narrow, so that in a moderate current it did not rise easily in spite of its light specific gravity, but it was operated without serious accident for sixteen years, when it was replaced by a steel bear-trap. Since then others have been built (of the "old" type, and of steel) at later dams. They are used as drift-chutes and to hold down the pool in quick rises, and are made up to 120 feet in length. The lower leaf has a double skin, and where necessary is used as an air chamber in

FIG. 238.—Sketch Illustrating the Design of the Chicago Drainage Canal Bear-trap

connection with an air compressor on shore. This arrangement is necessary in order to overcome the weight of the gate and to ensure reliable action against the dynamic effect of the current, which acts on the leaf much as described for drum wickets (see p. 648). The upper leaf has a single skin, usually with wooden sheathing. On Dam No. 6 the experiment was tried of using steel girders with wooden sheathing for both leaves, no metal skin plates being used. Two bear-traps were installed, each 120 feet long and with a vertical height of 13 feet 2 inches above the sill. (See illustrations on pp. 644 and 656.) It was believed that the girders would possess sufficient stiffness to counteract the tendency to warping which would follow the introduction of the water from one end. In actual operation, however, there occurred a considerable warping which generally became worse, one end rising from 3 to 5 feet in advance of the other and in 1909, after six years' use, both of the lower leaves gave way. The wooden sheathing on these two leaves was replaced in 1910 with steel buckle plates. With steel skin plates there has been found to be practically no warping, as the double plating of the lower leaf gives great stiffness,

Bear-trap Dam with Steel Sheathing.

View of Dam No. 1, Allegheny River, Showing the Two Bear-trap Weirs in Position and the Chanoine Wicket Pass in the Distance.

and this design is therefore considered much preferable. At Dam No. 6 the ratio of upward to downward force is as 100 to 80 and a head of about 3½ feet is needed to raise the gate. If air is used, the head required is 2½ to 3 feet. With a 9-ft. head the gate takes six to eight minutes to rise, and can be lowered in three to four minutes. At Dams Nos. 3, 4, and 5, built later, the ratio of forces is 100 to 66, and the gates rise in five minutes against a full pool and without the use of air.

Two bear-traps of excellent design were completed in 1903 at Dam No. 1, Allegheny River (Herr Island), at Pittsburg. They span openings of 90 feet each, and one has 12 feet of water on the sill, and the other 10 feet. The lower leaves

View of Bear-trap Weir Dam No. 6, Ohio River (1901), with Wooden Sheathing. The bear-trap is raised in position and seen from the downstream side. The bottom sheathing planks are yet to be put on. (See also p. 644.)

are of steel girders with double skins of ⅜-inch steel plate, made watertight, and the upper leaves are of girders with single wooden sheathing. (See Fig. 239 and accompanying illustrations.) The theoretical ratio of hydraulic upward and downward pressures is 100 to 80. The weight of the larger bear-trap is 95 tons in water and of the smaller, 82 tons. The lower leaf of the former, when filled with air by the compressor on shore, has a lifting force of about 49 tons in excess of its weight, while the corresponding excess force for the latter is about 14 tons.* With the assistance of air, the gates can be raised against a full current in five to twelve minutes each, although at first they needed a head of 6 to 8 feet, while with the

* See "Artificial Waterways," Transactions Am. Soc. C. E., vol. liv, Part F, and "The Improvement of the Ohio River," Transactions Am. Soc. C. E., 1908–9, by Major W. L. Sibert, Corps of Engineers, U. S. A., through whose courtesy the accompanying cuts are reproduced.

head alone the operation often took ten or twelve hours. Later the space between upper and lower leaves was reduced with boards, and it was then found that the gates could be raised under a $4\frac{1}{2}$-foot head without the help of air. The air has

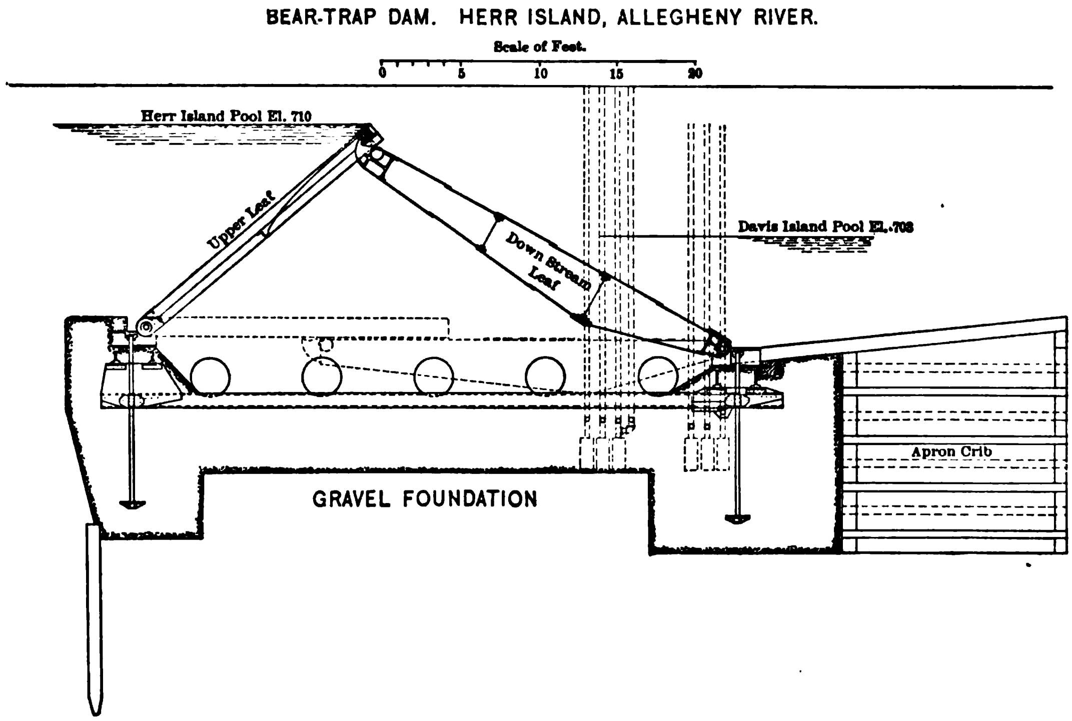


FIG. 239.

to be forced in at each operation, as it has been found to leak out entirely in about twenty-four hours. The lowering of the gates can be done in two or three minutes. Although the water is admitted at one end only, there is no appreciable warping of the gates.

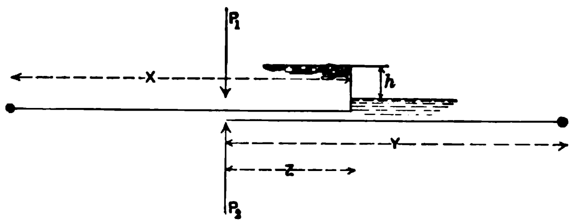


FIG. 240.

Calculations.—To determine the dimensions of the old-style two-leaved and of the Parker bear-trap, the following data have been deduced * (Fig. 240):

* Journal of the Association of Engineering Societies, June, 1896, from an article by Captain H. M. Chittenden, Corps of Engineers, U. S. A., and Mr. Archibald O. Powell. The accompanying curves are reproduced by permission from the same article.

OLD BEAR-TRAP.

Let X be the length of the upstream leaf;
Y the length of the downstream leaf;
Z the overlap of the two leaves;
P_1 the downward pressure;
P_2 the upward pressure;
h the difference of level between the entry and the exit of the operating flume, and
w the weight of a cubic foot of water.

Then $$P_1 = \frac{X \cdot Z - \frac{1}{2}Z^2}{X - Z} \cdot h \cdot w, \qquad P_2 = \tfrac{1}{2} \cdot Y \cdot h \cdot w.$$

If $P_1 = P_2$, $$Y = \frac{2X \cdot Z - Z^2}{X - Z}.$$

For the gate to move, P_2 must be greater than P_1.
Let the relation be represented by $nP_2 = P_1$, n being a proper fraction. Then

$$Y = \frac{2X \cdot Z - Z^2}{n(X - Z)}.$$

Combining this with an equation for the length of the gate, which is taken as unity, we find

$$Y = \frac{1 - \frac{n}{2}}{1 - n} - \sqrt{\frac{X^2}{1 - n} + \frac{1}{4}\left(\frac{n}{1 - n}\right)^2},$$

$$X = \sqrt{(1 - n)Y^2 - 2\left(1 - \frac{n}{2}\right)Y + 1}.$$

By substituting values of n between zero and 1 we find the corresponding values of X and Y. From this was platted the accompanying curve (Fig. 241), showing the apexes of the ordinary bear-trap when at full height. From it the proportions can be scaled, and it will be noted that the range is very wide. Thus if AB represents 12 feet by scale for a value of $n = 0.8$, GD will give the width of base required on a similar scale. H represents the depth on the hinge (as in Fig. 242) which is usually at about the same level as the sill. This curve assumes that the gate has no weight in water, so that if it is of metal recourse must be had to artificial means, as air or a greater head of water, to compensate for this.

To secure safe results it is best that n should not exceed $\frac{8}{10}$ or $\frac{2}{3}$. The ratio of $\frac{8}{10}$ has been adopted for several of the later Ohio River bear-traps, and in one or two cases $\frac{2}{3}$ has been used.

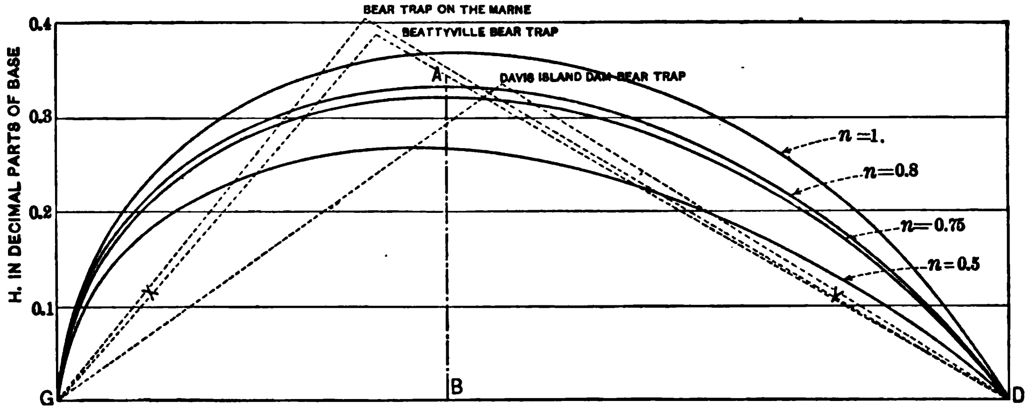


FIG. 241.—Old Bear-trap Curves.

Parker Bear-trap.—The general solution for this type is given by the equation

$$P_4-P_3\cdot\cos\theta-\frac{P_2\cos\gamma\cdot\sin\theta}{\sin\gamma}=0$$

the nomenclature being as shown in Fig. 242. From this as a basis the accompanying tables were deduced, and the curve of proportions platted (Fig. 243), the analysis being similar for the direct or upstream-folding gate, and for the reversed or downstream-folding gate.

TABLE OF PARKER-GATE PROPORTIONS

Φ	No Backwater.				$\frac{h}{H}=0.8.$				$\frac{h}{H}=0.6.$			
	X	Y	Z	H	X	Y	Z	H	X	Y	Z	H
10°	.994	.090	.084	.173	.....	.....	.....	.....	.....	.....	.....	.....
20°	.975	.185	.160	.333	.....	.....	.....	.....	.....	.....	.....	.....
30°	.946	.280	.226	.473	.....	.....	.....	.....	.855	.322	.177	.427
40°	.905	.376	.281	.582	.968	.353	.321	.622	.784	.430	.214	.504
50°	.854	.470	.324	.654	.896	.456	.352	.686	.702	.533	.235	.537
60°	.793	.560	.353	.687	.800	.558	.358	.693	.615	.630	.245	.533
70°	.724	.645	.369	.680	.677	.660	.337	.636	.524	.716	.240	.492
80°	.649	.722	.371	.639	.542	.754	.296	.534	.432	.793	.225	.425
90°	.568	.791	.359	.568	.437	.827	.264	.437	.350	.854	.204	.350

Φ	$\frac{h}{H}=0.4.$				$\frac{h}{H}=0.2.$				$\frac{h}{H}=$ limit.			
	X	Y	Z	H	X	Y	Z	H	X	Y	Z	H
10°	.....	.....	.....	.....	.....	.....	.....	.....	.9129	.137	.050	.159
20°	.....	.....	.....	.....	.....	.....	.....	.....	.8264	.267	.093	.283
30°	.782	.362	.144	.391	.748	.383	.131	.374	.7412	.388	.128	.370
40°	.704	.471	.175	.453	.666	.492	.158	.428	.6580	.497	.155	.423
50°	.622	.572	.194	.476	.587	.590	.177	.450	.5773	.590	.167	.442
60°	.538	.664	.202	.466	.509	.678	.187	.441	.5000	.683	.183	.433
70°	.458	.744	.202	.430	.433	.755	.188	.407	.4264	.758	.185	.400
80°	.386	.810	.196	.380	.362	.820	.182	.357	.3572	.822	.179	.352
90°	.322	.864	.186	.322	.302	.871	.173	.302	.2929	.874	.167	.293

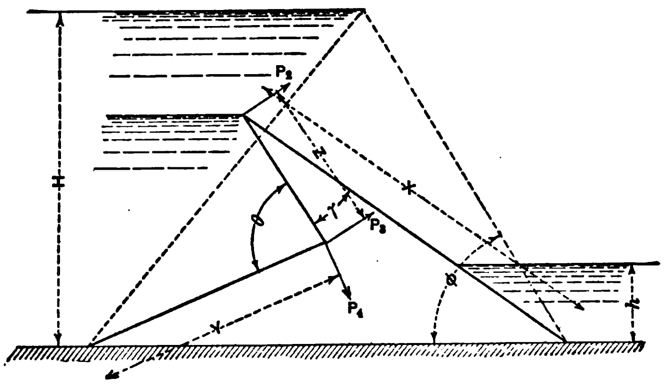


FIG. 242.

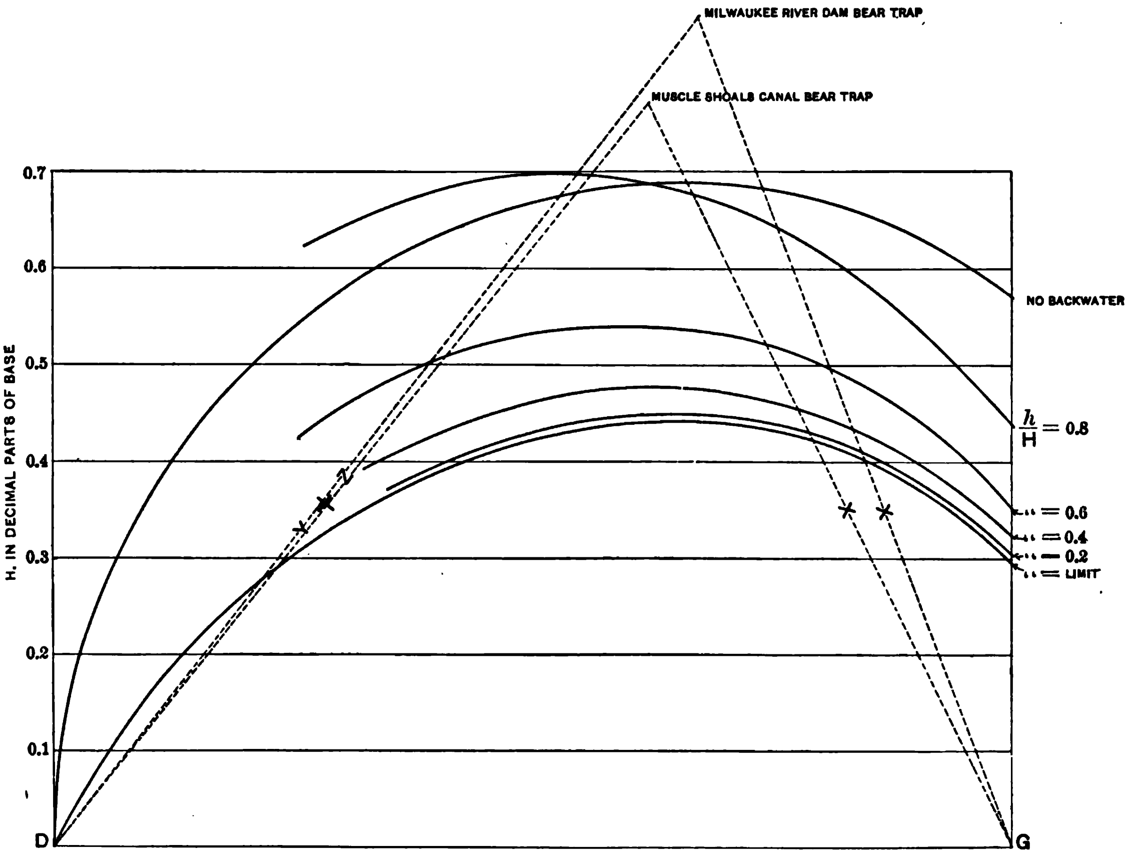


FIG. 243.—Parker Bear-trap Curves.

Lang Bear-trap.—The following is a table of data for proportioning a bear-trap of the Lang type, the letters and figure being the same as for the Parker gate.*

TABLE OF LANG-GATE PROPORTIONS

ϕ	No Backwater.				$\frac{h}{H}=0.2.$				$\frac{h}{H}=0.3.$			
	X	Y	Z	H	X	Y	Z	H	X	Y	Z	H
42½°	.....	.....	.....	.....	.....	.....	.....	.....	.706	.485	.191	.478
45°	.....	.....	.....	.....	.....	.....	.....	.....	.691	.508	.199	.490
47½°	.....	.....	.....	.....	.715	.512	.227	.528	.670	.534	.204	.494
50°	.729	.521	.250	.558	.697	.536	.233	.534	.644	.562	.205	.493
52½°	.714	.544	.258	.567	.668	.564	.232	.530	.612	.591	.203	.486
55°	.672	.576	.249	.550	.637	.592	.229	.522	.584	.618	.202	.479
57½°	.635	.608	.242	.536	.616	.616	.232	.520	.559	.642	.201	.472
60°	.611	.630	.241	.529	.590	.640	.230	.512	.536	.665	.201	.463
62½°	.585	.655	.240	.519	.564	.664	.228	.500	.....	.....	.....	.....

ϕ	$\frac{h}{H}=0.4.$				$\frac{h}{H}=0.6.$				$\frac{h}{H}=.75$ to 1.0.			
	X	Y	Z	H	X	Y	Z	H	X	Y	Z	H
42½°	.678	.499	.177	.457	.660	.510	.170	.446	.676	.501	.177	.456
45°	.651	.529	.180	.461	.632	.539	.171	.447	.653	.528	.181	.462
47½°	.627	.556	.183	.462	.610	.565	.175	.451	.632	.553	.185	.466
50°	.600	.584	.184	.461	.590	.589	.179	.453	.611	.578	.189	.468
52½°	.576	.609	.185	.457	.570	.612	.182	.453	.590	.602	.192	.469
55°	.551	.634	.185	.452	.551	.634	.185	.451	.570	.625	.195	.467
57½°	.527	.658	.185	.444	.529	.657	.186	.446	.549	.647	.196	.463
60°	.505	.680	.185	.438	.510	.678	.188	.442	.529	.669	.198	.458
62½°	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

The curve (Fig. 243) and these tables assume, as with Fig. 241, that the parts have no weight when submerged. With metal gates, the ratio of H and the base must be made considerably smaller, or special means must be employed to overcome the weight, as previously mentioned.

The proportions can be scaled from the curve as explained for Fig. 241, or can be taken from the tables by the following method. The first column, headed "no backwater," gives the proportions of leaves for gates to be used as weirs only, where they will not be subject to any back pressure from a pool below; the columns following give those for gates subject to such back pressure of constant amount in each particular case; and the column "limit" in the Parker table gives the proportions where a gate is subject to varying heights of backwater, a condition generally obtaining in rivers. To find the theoretical lengths of leaves for a given height of water H, the angle ϕ (Fig. 242) made with the horizontal by the long leaf X is first selected. This will usually lie between 45° and 60°, for an economical length of base, as shown by the curves (Fig. 243). The corresponding values of X, Y, and Z are then taken from the tables, and their required length in feet deduced from the proportion between the table value of H and its actual

* Transactions American Society of Civil Engineers, June, 1898. Tables calculated by Mr. T. C. Thomas.

value in feet. Thus if $\phi=50°$ and $H=12$ feet, we find from the "limit" column for a Parker gate that $X=.577$, $Y=.590$, $Z=.167$ and $H=.442$. Then if $x=$the required value of X in feet, we have

$$\frac{x}{X}=\frac{12'}{H}, \quad \text{or} \quad \frac{12'\times X}{H}=\frac{12'\times.577}{.442}=15.67 \text{ feet.}$$

Similarly we find $Y=16.18$ feet and $Z=4.53$ feet. The length of the base, or the theoretical distance between the upstream and downstream hinges, is found in each of the three types of bear-trap here discussed from the equation $X+Y-Z=1$, and for this example is 27.32 feet.

The height H is measured from the hinge of the gate, and will usually be a little more than the depth of water on the sill.

The dimensions may be approximately checked from the curve (Fig. 243) by drawing a line from G, inclined to DG at the angle ϕ, and noting the point where it strikes the corresponding curve. By joining this point with D the outline is obtained of the gate when fully raised. It will be observed that the outline shown on Fig. 243 of the Milwaukee River bear-trap, which was one of the earliest of the Parker type, indicates an insufficient width of base, and in point of fact the two gates there were always somewhat troublesome in operation, requiring to be heavily loaded with iron to ensure their lowering. This of course required an additional head for raising, which was fortunately available. The leaves X, Y and Z (Fig. 242) must be so placed that when the gate is bedded there will be no contact between them except at the hinges, and ample space must be left for the proper circulation of the water. This condition may require a slight variation of the theoretical lengths of the leaves, as the lower hinge of Y must be placed on a lower level than that of X.

The method of proportioning indicated above applies equally to the direct and to the reversed Parker type, the angle ϕ being measured in both cases from the long leaf X. In the reversed type, however, it is necessary to provide anchor chains or bars connected to the foundation and to the common hinge of Y and Z, in order to prevent these leaves from being forced out by the internal water pressure.

Choice of Type.—The old bear-trap possesses the merit of being the simplest of all three types, having only two moving parts where the others require three or more. This is a great advantage with this class of dam, as breakage or injury in any portion is liable to put the gate out of operation, and owing to its size repairs may be costly and slow. This type appears also to be less liable to be injured by ice or drift, and for these reasons it was considered by the United States Army Engineers, after some twenty years of trial, to be best adapted for use on the upper Ohio and similar unruly streams. It requires a greater width of base than the

Parker or the Lang, but in view of the simpler design of the gate itself, and its greater capacity for hard service, the additional cost of foundation may prove of less importance than would at first appear.

The relative merits of the Parker and the Lang types are not well defined. The former is probably the better where a bear-trap is to be used as a lock-gate, as it provides a tighter chamber and possesses a greater efficiency for a given length of base. The Lang has proved very successful where properly proportioned, but the Parker appears to have had a more extended application. Three gates of the Lang type, one of which was 80 feet long and supported a head of water of 14 to 16 feet, were built in 1890 at Nevers, Wisconsin, by a lumber company, and in thirteen years needed but little attention beyond ordinary repairs. The idler or sliding leaf shown on the cut of the Parker gate (Fig. 234, p. 652), and used as a precaution against drift, can be omitted with advantage in many cases, but on the Lang gate it must be used. A rush of water has a strong tendency to lift an idler which points upstream, and a Lang gate on the Chippewa River was wrecked from this cause. The current caught the idler and swung it up, tearing it loose and breaking the main gate.

The relative merits of the direct and reversed Parker types are also somewhat uncertain, and appear to lie in the question of the passage of drift and overflow. The reversed type with the long leaf upstream affords less opportunity for the catching of drift and ice, while the direct type gives a better downstream slope for the passage of the overflow.

Design of Details.—Several practical points must be observed in designing the details for a bear-trap, chief among which are the following:

All parts should be made as simple as possible, and with a view to their easy removal and repair. Owing to the great size of this type of dam, repairs are liable to be difficult and costly, and an accident which might prove insignificant to the small units of a wicket or needle dam, may prove very serious to a bear-trap. (See p. 665.) To facilitate repairs, arrangements should always be provided for temporary cofferdams, such as small trestles hinged to the masonry, or sockets in which uprights can be placed to support plank, etc. Ringbolts at intervals along the crest are convenient for hoisting the gate in its recess during repairs, and manholes for inspection should be arranged for.

The question of leakage is important, as, if the joints are loosely fitted, there will be a considerable loss of pressure, and the operation of the gate will be interfered with in consequence. The supply and discharge culverts should be of ample size to compensate for any such loss of pressure, and where practicable it is an excellent plan to have the supply culverts commence some distance above the dam. By this provision an additional head is frequently obtainable, as the surface slope of the water steepens rapidly as it nears the crest when the dam is lowered or

partially lowered. Data as to the size of flumes are meagre, but for a gate 45 feet long by 14 feet high, the net supply and discharge areas should apparently be not less than 10 square feet each. The Louisville gate (Fig. 237, p. 653) has a little more than 0.25 square foot of culvert to 100 cubic feet of contents when raised. The flumes should be built without sharp corners so as not to reduce the head, and the upstream end should be protected by a grating so as to keep out drift. This end should be kept high where practicable so as to avoid the heavy sediment which travels along the bottom of a river.

The gate, if of timber, should be made a little heavier than water, so that it will not tend to float up.

One of the difficulties met with in long gates, especially if of timber, has been a tendency to warp, as the water has usually been introduced from one end only. This tends to cause one end to rise before the other, and to twist and strain the leaves. Racks and pinions have been tried to remedy this, but without much success. The most rational method of guarding against it would be to provide a culvert running the whole length of the foundation, and to admit the water simultaneously through several openings, thus equalizing the pressure. This was done on the bear-trap of the Chicago Drainage Canal, and worked well. The only design which appears to be practically free from this warping is that where the lower leaf is composed of steel, of deep girders covered with double plating, as described on p. 654.

It was found desirable on one gate to form the hinges of long pieces or shafts, replacing the pins used in the original construction, as there appeared to be a tendency to bind when the latter were used. Short pins have worked satisfactorily in other cases, however, where sufficient clearance was provided in the pin holes to allow for defects in setting.

Especial care must be taken to guard against scour at the downstream side of the foundation and along the adjoining piers or abutments, as the rush of water when the gate is lowered under any head is tremendous, and has been known to carry off heavy riprap and to scour holes 30 feet deep in a gravel foundation in less than an hour. In one case the scour during one season extended to bed-rock, 42 feet below lower pool level. In other cases deep scour has occurred along the upstream face also. (See pp. 540 and after and 549 for description of protection, etc.)

Where idlers are used (see Fig. 234, p. 652) rollers must be introduced at their ends, or the friction will interfere with the operation of the gate, and small openings must be provided to equalize the pressure of the water on both sides. Rollers must also be used at the end of the lower leaf of the old bear-trap for similar reasons.

With the direct Parker type, in which the folding leaf is on the upstream side (Fig. 234, p. 652), the internal pressure when the gate is erect must not be allowed

to act upon this leaf to an extent which will force it outwards, or serious damage may ensue. This condition is not liable to occur, however, except where a special head of water is used. With the reversed type (Fig. 235, p. 652) this danger is inevitable, as any internal water pressure tends to force out the folding leaf, and it has to be guarded against by using chains fastened to the hinge and to the floor. Their length must be such that they will halt the gate before the two parts of the folding leaf come into line, or the latter will tend to act as a direct strut, holding up the upstream leaf and preventing the gate from lowering.

In both the direct and reversed Parker bear-traps the leaves X and Y (Fig. 242, p. 660) should be parallel when lowered, or should make an obtuse angle facing downwards. If placed otherwise, the position of the leaf Z, holding the ends of X and Y at a fixed distance, will tend to cause the latter to oppose each other's movement at the initial raising or final lowering. With all types of bear-trap there must be allowed room for the water to reach freely all parts on which it is to act when the gate is to be raised.

Remarks.—The bear-trap, while one of the first developments of the movable dam, has had only a limited application. This has been due partly to the fact that early experience with them was discouraging, the theory of their porportions not being then understood, and partly to the expense of construction, as the gate and foundations are costly, and have in some cases involved more than twice the outlay for the Chanoine wicket dam which formed part of the same structure. They are very useful as weirs on streams of sudden freshets and much drift, and have proved of great value on the movable dams of the upper Ohio and Allegheny rivers, where the floods occasionally rise at a rate of more than a foot an hour. On the Ohio dams, where the movable portions are in some cases 1000 feet in length and over, it is imperative to have a quick means of keeping down the pool level and passing drift until part of the wickets can be lowered, and bear-traps have proved fully equal to these duties.

The principal objection urged against the type, aside from its cost, has been that it consists of only two or three pieces, and an accident to one part may result in injury to the entire gate. The Louisville bear-trap (Fig. 237, p. 653) a few years after its completion had many of its anchorages broken and sustained much injury to other connections by reason of a small piece of metal catching between the end of the downstream leaf and its cast shoe. There was no room for the piece of metal to escape, and the resulting leverage was so enormous and the gate so rigid that many fastenings gave way before the strain was relieved. The piece was supposed to have been left in the recess during construction, and to have been finally carried to its vantage point by some eddy of the operating water. A bear-trap on the Mississippi River near Minneapolis, in use by a commercial firm, was interfered with by saw-mill refuse getting into the moving parts, and by ice forming on the downstream

leaf and causing the gates to lower, and it was finally replaced by an ordinary sluice gate. At Dam No. 2 near the same locality the bear-trap sluices, which were of the reversed Parker type, were injured in 1907 by a deposit of mud and trash 2 feet in depth which had settled inside and prevented the leaves from bedding properly. All the holding-down rods to which the check or anchor chains were attached (Fig. 235, p. 652) were bent or broken in consequence.

On the other hand, the wooden bear-trap at the Davis Island dam of the upper Ohio, where the conditions of drift and sediment are both severe, performed its functions without serious accident for about sixteen years, by which time it was worn out and was replaced by a new one of steel and timber. Experience with the first gate, broadened by that with later ones on this and other rivers, has shown that where conditions require a rapid means of regulation, and the operation can be made certain by artificial means, as by using compressed air in one leaf (see description of the Allegheny bear-trap, p. 656) or an independent head of water, as from a storage tank, the bear-trap is very useful and its liability to accident probably no greater than with other types of automatic dams.

The head required to raise a wooden gate should not be over about 6 inches in still water. On the Marne, owing to faulty proportions, a head of 2 feet was required, which was obtained by a row of Thénard shutters just above the gate. Where the head is not obtainable from the natural fall of the river the raising may be secured by a pump and ample reservoir tank on shore, or by using compressed air as just mentioned. The dynamic effect of the current passing over a gate when rising is considerable, and may neutralize a foot or two of the head, as described for drum wickets on p. 648.

Cost.—Data as to the cost of bear-traps are meagre. The two steel bear-traps at Dam No. 4, Ohio River, each 93 feet long, cost $350.50 per lineal foot for the fixed parts and $340.75 for the movable parts, or a total of $691.25 per lineal foot. The bear-trap on the Marne (p. 651) cost $488 per lineal foot.

ROLLING DAMS.

About 1900 a new type of movable dam was patented and introduced in Germany, although the principle had been advocated previously by several different engineers. It consists of a braced metal tube or cylinder, resting on a masonry foundation and provided with heavy toothed racks at each end fitting into corresponding racks on the piers or abutments, as shown on the accompanying illustrations. The tube is rolled up by powerful machinery working from one end only, and revolves on the racks, allowing the water to pass underneath. The top of the masonry foundation is usually fitted with a wooden sill which can be cut to a close fit with the tube, and tightness is secured at its ends by causing them to bed against a flat band of hemp or similar material, or by using loose stanching

Rolling Dam at Schweinfurt, Showing Rack and Operating Machinery.

The Larger Schweinfurt Dam, During Erection, Showing How Dam is Raised to Pass Floods.

ROLLING DAMS AT SCHWEINFURT, BAVARIA.

rods. The first dam of this type was built in 1900–1901 on the Main at Schweinfurt, Bavaria, by its designer M. Carstanjen, and closed an opening 108 feet long by 6.6 feet high. As it proved successful, a second one was built in an adjoining branch of the river of 59 feet clear opening and 13.5 feet high. These dams have proved very satisfactory and have withstood unharmed the attacks of floods and ice gorges, being kept in place all winter for the sake of water power. The leakage is extremely small. In the longer dam the two ends rise slightly in advance of the center during motion, as there is naturally a sagging, and it was found also that the effect of a hot sun on the tube when bedded caused a distortion sufficient to permit leakage at the center. This is guarded against in new designs by allowing for a variation of length. In some of the later dams the tube is less in diameter

The Larger Schweinfurt Dam, Lowered in Place.

than the depth on the sill, and carries a braced steel plate or shield fitted to its upstream side, and which supports the water like a flashboard.*

A considerable number of these dams has been built since 1900, although up to 1910 the original ones just described were the largest as regards diameter and length considered separately. The largest of combined dimensions was one in Sweden, with a diameter of 11½ feet and a length of 105 feet, two spans of these dimensions being used.

While they are somewhat expensive owing to the boiler-like construction, and heavy in operation owing to the absence of counterbalances, they are strong and reliable and can be operated at all seasons. For perennial water-power purposes they have proved equal if not superior to any other type hitherto used.

* See Transactions Am. Soc. C. E., International Congress of Navigation, 1904, paper on Rolling Dams by Professor K. E. Hilgard, through whose courtesy the accompanying cuts are reproduced.

CHAPTER X.

ACCIDENTS TO STRUCTURES, ETC.

IN the following pages are described some examples of typical accidents and failures in various countries. Some of these happened a generation or two ago, others are comparatively recent, and for this reason the names of many of the localities have been omitted.

Flanking.—This is one of the worst and most costly of mishaps to locks and dams, and consists in the river cutting around or "flanking" one end of the structures. It has been, however, by no means an uncommon occurrence, although it has apparently been due almost always to lack of sufficient care in design or in construction. The following are typical examples of the causes and effects.

In constructing a crib dam across a river of much variation of discharge, the builder, deceived by a long period of low water, put off completing the filling behind the abutment which had been otherwise finished, and was built on rock. A rise came, and the dam itself being almost completed, a considerable head of water was caused, and the current eventually topped the uncompleted fill behind the abutment. In a short while a channel was washed out there, rapidly widening until the entire flow of the river was passing through it. Nothing could be done to remedy matters until the succeeding period of low water, when the gap was closed with rock-filled cribs and the river turned back over the dam. Fifteen years later the gap had become filled with sediment from floods and was grown over with brush until little of the break was visible.

Some years later the same river flanked a lock further upstream. This was due to the omission of some of the riprap protection originally intended to guard against such erosion. The first high flood after the completion of the works rose to about 4 feet over the lock walls, and there was at that time a difference of several feet between the pools, the dam being of the fixed type. This produced currents which cut out the backfilling, beginning at the lower end of the lock, and within twenty-four hours the crest of the dam was dry and the entire flow was passing behind the walls. Lock houses and other property were washed in, although the lock itself, being on rock, was unhurt. Repairs were made during the next low-water season by building a fixed dam, with a crest 5 feet above that of the original dam, across the gap, so that the lock now stands near the middle of the river. The scoured area above the new dam began filling with deposit at once, and eight

years later its upper end had filled up almost to the pool level, gradually sloping downward from thence towards the new dam. (See also paragraph immediately preceding.)

Another case occurred many years ago with a crib dam built to close a new cut-off channel made by a large river, and which had begun to change seriously the water levels and the regimen of the stream. Owing to disputes with a land owner, the dam had been almost completed before the abutment was begun, and a flood which occurred soon after cut a new channel around the dam and through the unprotected bank. As navigation was fast disappearing, owing to the competition of railroads, repairs were never made, and the dam now lies buried under sediment and vegetation, while the river flows through the new channel at one side.

A more recent example was that of a crib dam on a river subject to high floods. The top of the abutment was low, being about level with the crest of the dam, and stone paving was used to protect the slope of the bank behind it. Below the abutment cribwork was used at the toe of the bank with more paving. All the structures were on gravel. Three or four years after the completion of the work, when a flood was running 8 or 10 feet deep over the crest of the dam, the abutment settled suddenly and went out of sight, and in a few minutes a large volume of water was flowing around the end of the dam and cutting rapidly into the bank, threatening valuable property and the tracks of a trunk-line railroad. Riprap was dumped over the bank, and as soon as the flow through the new channel had drawn down the water to a foot or two above the crest, dynamite was placed in the dam and a length of nearly 600 feet of the upper portion was blown out to a depth of about 12 feet. This turned the main current back over the dam, and on the recession of the flood the eroded bank was temporarily secured. Later on a new abutment was built near the new bank line and the dam was extended and repaired. The cause of the break was supposed to be either a leak under the abutment or damage from drift to the cribwork downstream, which loosened the paving and allowed the current to attack the earth beneath.

On another river a crib dam 325 feet in length was built at a point where the natural width was nearly 500 feet. This was done for reasons of economy, and the top of the abutment was placed 2 feet above the crest of the dam and connected to the original bank by a slowly rising wall 128 feet in length. In floods this wall was intended to act as part of the dam, the space downstream of it being backfilled and protected by grouted riprap. The foundations were gravel and sand, and the abutment rested on piles. About a month after the completion of the work a flood rose over the abutment (the difference of water level above and below being about 12 feet) and attacked and undermined the toe of the riprap below. The erosion worked its way upstream rapidly, although much riprap was thrown in in an attempt to stop it. The water eventually cut back to the abutment, under-

mined it and turned it upside down, and between midnight and dawn had cut out a new channel around the dam 200 feet in width. The break was closed during the following summer by extending the dam about 345 feet and building a new abutment at its end.

The foregoing are typical examples selected from many on record, and suffice to show the need of careful design and construction of those portions exposed to flood currents. The cost of repairing such breaks has varied from $50,000 to $250,000.

Undermining. (See also the preceding paragraphs on "Flanking.")—The undermining of a dam appears to be more usual from the downstream than from the upstream side in river work, presumably for the reason that the head of water is comparatively small and that the danger in the latter case is so obvious that precautions are naturally taken against it. Two instances have been referred to on p. 374. In another example, with a head of only 3 feet, no protecting downstream apron or upstream sheet-piling was provided, although the dam was built on gravel. The reaction began to erode under the toe and gradually worked back until the upper pool found its way under the dam. A tendency to undermining from above has often been found at the earlier dams and locks where built on light material, and has frequently been due to a lack of sheet-piling or to the sheet-piling having become worn by leakage. Leaks under such circumstances may exist for some years without being observed until repairs are made or some accident occurs. In one case during repairs to an old lock it was found that a leak had undermined part of the wall until a man could stand upright and walk about under it. As the masonry was on piles nothing unusual had been noticed. In another case, without piles, the natural clay under one of the gate recess walls had seeped out for a depth of a foot or more until a length of nearly 30 feet of wall had no support except that given by the masonry abutting each end. In a third case settlement of the end of a wall had continued very slowly through many years, and was found to be due to a tiny spring which had gradually carried out the supporting material.

If a leak or swirl appears above a lock or a dam with muddy water coming out below, no time should be lost in trying to close it. If a dredge is at hand, have her dump any material available into the swirl and try to stop it quickly. If a dredge is not available, throw in one-man riprap so as to make a bottom layer, and with this and on it throw in spalls or gravel, and then earth. The riprap if sufficient in amount usually stays in place and breaks the rush of water, and the gravel catching in it will check the flow still further, and finally the earth will close the interstices of the gravel. If no riprap is at hand, use bags of earth, pieces of shrubs, or anything that will catch temporarily at the bottom of the hole and give a bed for the gravel. If gravel is not available, it may be necessary to break up some of the riprap into spalls and to throw it in with the larger

pieces. The object to be attained is first to get a bed of material at the bottom of the hole which will be of pieces large enough to catch there and break the rush of water; the small stone or gravel thrown in next is carried into the spaces in this material and reduces the leakage still further; and lastly the earth seals up the spaces in the gravel and cuts off the flow. Such places should be carefully watched thereafter lest the leak break through again.

While in many cases there is little warning in advance, the danger can often be averted by quick action upon discovery. A case in point is one where the engineer, having just completed a crib dam, noticed a little muddy water appearing below the abutment after the pool had filled. He placed a watchman to report any increase in the quantity of discoloration or any settlement in the fill behind the abutment, and late at night the watchman discovered that the fill was settling. An examination showed a large leak, and men and teams were immediately obtained and brush was cut and hauled to the site from a neighboring field and placed in the water just above the abutment, and weighted down with clay. The leak was closed in a few hours and has never reappeared, although the dam itself has been rebuilt because of decay.

On some dams on sandy foundations a gradual settlement of the upstream backing has been noticed, indicating a seeping out of the sand beneath. In such cases the backing usually has been leveled up again every few years or oftener, a mixture of gravel and clay being found the best material, although clay alone will give good results. This method appears to prevent conditions from becoming worse.

Occasionally leaks are met with which appear to cause no damage and which would be very expensive to close; in such cases they are often allowed to continue, although kept under careful watch. One instance occurred at a lock on a foundation of hard limestone. Several seams had been uncovered at the time of the construction, running across the base of the river wall, but they were not all properly stopped up. After the completion of the work, when the lock chamber was filled for the first time the leakage through the seams was plainly visible outside the wall, and as soon as the gates were closed the water surface fell several inches in less than ten minutes. An attempt was made some time afterwards to unwater the chamber, but without success, and at a still later period a blanket of gravel and mud was placed over the rock floor to a depth of 6 feet (the rock lying about 10 feet below the top of the lower miter-sill) and another attempt at unwatering made. This also failed. An investigation was then made by a diver, and he found a seam about 14 inches in width with a depth in places of 18 or 20 feet, and running along the chamber for more than 100 feet. The last attempt at unwatering showed that the seam had connection with both the upper and the lower pools, as when the pumps were stopped, after reducing the surface to about lower pool level, the

water rose at a rate of nearly 1 inch per minute. The size of the chamber was 52 feet by 180 feet, and the low-water lift between pools was 18 feet.

In another instance, an old lock had been filled in, a new one having been built alongside. Both were on rock. A leak developed through the filling, entering, it was believed, in some of the old stone-filled guide cribs at the head of the old lock, and escaping at the lower end of the grade. For some years attempts were made to close it by occasionally dumping material on and around the old cribs, and while this method was successful on one or two occasions the leak always reappeared within a short time, and the attempts were finally abandoned.

In a third instance, with a lock on piles driven in a sandy gravel, a leak appeared on the outside of the river wall immediately after the completion of the dam. As it was running clear, and as there was then no indication of where it was entering, developments were awaited. Within two or three weeks a small sink-hole appeared in the filling just above the upper wing wall of the lock and behind a stone-filled guide crib, showing that the water had found an entry there through the wooden sheet-piling of the foundation, and was passing under the timber floor of the lock and escaping through the sheet-piling along the outside of the river wall. The conditions were such that no thorough remedy appeared possible without removing part of the guide crib and excavating to the foundation of the wall—an expense which did not appear justifiable. It is stated that the leak has now been in existence for more than ten years, without causing any apparent damage beyond the reappearance at long intervals of the small sink-hole before mentioned.

On a neighboring river at a dam on a limestone foundation and with the abutment built against a limestone cliff, there was a very copious leakage behind the masonry, the water apparently passing through the partings between the strata. One of the leaks appeared large enough to fill a 12-inch pipe. Various efforts were made to cut off the flow by dumping material above the abutment, but to little purpose. The only apparent disadvantage from the leakage was that in low-water seasons it drew down the upper pool too far; there was no evidence of other harm. As far as is known the leaks have been flowing since about 1840, when the dam was originally built.

Where, however, a structure is built on a foundation which can be easily eroded, such as sand, the continuance of a leak may bring about dangerous results, and if the flow cannot be stopped conditions should be watched closely.

Scour.—This is a fruitful cause of trouble with locks and dams, and its effects may vary from the slow erosion of the bank to the destruction of a lock wall. It is especially to be guarded against below locks and fixed dams on foundations other than rock. (See pp. 414 and 549 and after.) There have been examples on rivers subject to high floods, and canalized with fixed dams with insufficient widths of

apron and too little riprap, where the water scoured out the gravel bed to more than 30 feet below the original level, leaving part of the dam standing on the edge of the pit. Cases are also on record where the material along lock walls has been scoured away by the discharge from the dams until the walls (which were on piles) moved visibly towards the river each time the chamber was filled, the masonry having parted in the walls and along the floor. In other cases where no piles were used and the masonry rested on the natural material, part of the walls have fallen over into the river. One unusual instance occurred with a wall built on rock about 1840. About fifty years later the river wall suddenly fell over under the pressure of a full pool in the chamber, allowing the water to pour by the upper gates and over the ruins into the river below. When repairs were made it was found that the current from the dam (which was of the fixed type, of about 16 feet lift and at the head of the lock) had cut into the rock under the bottom course of masonry and gradually worked in until the wall became unstable. Details as to the actual cause of the wearing are lacking, but it is possible that the rock was seamy, and that as the softer material was washed out pieces of the rock became loose and were carried away, thus gradually enlarging the cavities.

Similar dangers from scour exist of course at the abutments, but engineers appear to have taken more precautions there, as records of failure are comparatively rare.

Where scour has produced dangerous conditions, matters are usually remedied by throwing in riprap in pieces of $\frac{1}{2}$ to $1\frac{1}{2}$ cubic yards or larger, or by sinking stone-filled cribs along the face of the dam or wall. Additional cribs or riprap may have to be added from time to time, and the whole should be carefully watched.

Scour is particularly to be guarded against with unfinished structures on sandy or light foundations, as the contraction of the cofferdam usually causes strong currents and eddies in floods which may undermine exposed ends. The cofferdam itself is usually a sufficient protection to any unfinished work inside it, but work outside should be well secured by riprap. In one case, where the work was prolonged for several years owing to lack of funds, considerable portions of the dam (built on sandy material, but on piles driven 20 feet below the original bed) were undermined and carried away by various floods, as were also two piers which were to form part of the finished structure. One of these sank out of sight during a rise, and soundings made later showed that the water had scoured below the original foundation level to a depth of nearly 20 feet.

Sliding.—Failure by the horizontal sliding of the structure appears to be rare with locks and dams, although not uncommon with retaining walls. One example occurred with a movable dam having a concrete foundation built on shale. This class of rock usually consists of very thin layers and possesses very little cohesion.

The concrete was from 3 to 4.6 feet in thickness. The failure occurred under a head of 10.7 feet, the concrete sliding bodily downstream, the bedrock, as far as could be told at the time, having apparently sheared off or crumpled up.

Other examples are referred to on pp. 376 and 378.

Accidents to Floors.—Accidents to floors are usually due to undermining from leakage or to underpressure, although they have occasionally resulted from floods scouring through the lock chamber during construction. In one such case where the lock was on gravel and the floor unprotected, an adjoining cofferdam deflected the current in a way that caused a rapid flow through the chamber (the lock gates not being in place), washing out the floor and undermining portions of the walls.

Trouble with undermining from leakage is often met with in old locks which have wooden floors, as the timbers become greatly worn in the course of time, allowing the water to run through the joints.

An unusual accident from underpressure occurred at a lock on a slate rock foundation. The slate was somewhat seamy, with occasional small springs appearing through it. The chamber was accordingly floored with a foot of concrete in which drain holes were supposed to be placed for the escape of the springs. Whether they were actually provided or whether they became clogged is uncertain. Not long after the completion of the floor, the lock being still unfinished, the underpressure pushed up and cracked the concrete along the center for about half the length of the lock, raising the middle about a foot and breaking the floor into two large slabs. The portions close to the walls did not appear to be disturbed.

Another case, which illustrates the uncertainties of underpressure, occurred in 1903 at the small lock at Mirowitz on the Moldau. This lock was built on gravel, without piles, and was situated so that its foundations were subjected to the pressure from the upper pool, amounting to a net head of about 13 feet. The floor was not arched, and was composed of about 2 feet of concrete, laid directly on the gravel. No damage occurred for some months, although the weight of the floor was much less than the theoretical thrust of the upward pressure. Finally a small portion was forced up close to a drainage ditch which had not been properly filled, and which was in consequence believed by the engineers to be largely responsible for the accident. Repairs were made by replacing the broken concrete and drilling weep-holes in the floor about 15 feet apart to relieve the pressure. No further trouble has occurred.

Instances of injury to wooden floors resulting from the nature of their construction have been given on p. 410.

Accidents to Lock Gates.—Lock gates nave occasionally been injured or carried away by breaking loose during high water. It is customary on the advent of a flood to leave one pair of gates open, usually the lower one, each leaf being locked

to the wall. Where the dam is movable both the upper and lower gates are frequently left open (see p. 486). Below fixed dams of high lift, and in some locations where the lift is moderate, there is more or less reaction or "backlash" in the lower lock entrance during freshets. This carries deposit into the lock and renders the closing of the lower gates difficult after the flood has subsided. For this reason lock-tenders are disposed to leave the lower gates closed as long as possible, or until they are about to be submerged, when each leaf is swung into its recess and fastened to the masonry. If the leaves are properly fastened, so that there is no oscillation, accidents rarely occur, but occasionally heavy drift becomes entangled in the fastenings and wrenches them from their anchors. Once loose the leaf will swing to and fro against the miter-sill with great violence, sometimes injuring it and generally breaking the gate anchorage and going out with the current. If, however, the leaf is carried away immediately after becoming loose it usually "jumps" the sill without injuring it.

The method of locking the lower gates together at the toe, thus leaving them mitered against the sill in order to prevent deposit in the lower end of the lock chamber, has led to many disasters to gates where the dams have been of the fixed type. The water below the dam is usually very rough and surging, and the gates are in consequence swayed back and forth, slightly but continuously, until the fastening is loosened and finally broken, usually resulting soon after in one or both leaves becoming dismounted. This type of accident does not ordinarily result in much damage to any part beyond the breaking of the main anchor bars, although occasionally the miter-sill is loosened or splintered, when the injury is more serious. There is, however, always a delay to navigation during the replacing of the lost parts, and when the gate or gates are not recovered, as has sometimes occurred, this may become serious.

A more frequent cause of injury results from boats striking the gates on the downstream side when they are closed and supporting a head of water. This usually is followed by the loss of one pair of gates and not infrequently of both pairs. One such mishap on the Welland canal in Canada broke loose three pairs of gates, the pair at the upper end of the lock being struck by a boat, and the pair at the lower end, as well as the upper gates of the lock immediately below, which were being closed, being caught in the sudden rush of water and torn from their hinges.

Accidents of this nature appear to have been rare with river locks, but they are not uncommon on canals. Even as large a waterway as the Manchester Ship Canal has not been exempt, for in 1906 the upper gates of the Irlam lock were carried away and the upstream level was emptied. The damage in such cases results from the boat striking one of the leaves and forcing its toe upstream, usually not more than 2 or 3 feet, but far enough to leave its mate without proper support at the miter. The head of water accordingly displaces the latter

leaf; a similar pressure on the first leaf drives it back a moment later (moving the boat before it), and the result is that both leaves are forced over and torn loose. It has occasionally happened that a boat has struck in a manner which allowed the leaves to barely miter again on their return and no damage has resulted, and one case is recorded where the blow took place exactly in the center, and the leaves came back and mitered perfectly. This happened in 1910 at the Poe lock at St. Mary's Falls, Michigan, an unloaded steamboat nearly 400 feet in length striking the gates and forcing them upstream 3 or 4 feet before the pressure drove her back and closed the gates again. The mishap was due to an accident to the machinery as the boat was entering the lock, the engineer being unable in consequence to stop her promptly.

The foregoing type of accident occurs only when the gates are struck from below. When struck from above serious damage rarely occurs. One case is recorded on a Canadian canal where a lake steamer of some 2000 tons burden drove on to a pair of solid wooden gates from above, breaking several of the upper beams without forcing the gates loose, then slid back without doing further injury.

When gates are broken loose in a manner which destroys all closure, as above described, there results, of course, a tremendous rush of water through the lock. With a canal having a short pool above, the water usually runs out without doing much damage to the banks or structures, but with a long pool the damage may be considerable. One of the most notable breaks of this character occurred in June, 1909, at the Canadian lock at Sault Ste. Marie (St. Mary's Falls), Michigan, which connects Lakes Superior and Huron. The chamber at the time was full and the upper gates were open. One of the lower gates was struck by a steamship at a moment when another ship was entering the lock on her way down and a third ship was moored in the chamber awaiting the maneuvers. Both lower leaves were carried away; the three vessels were swept out into the river, one of them sinking shortly after; and the upper leaves, which although open were not fastened back, were swung out by the current and were also carried away, and the waters of Lake Superior began to flow through the canal and the lock chamber under a head of about 19 feet. Upstream of the lock there was fortunately a swing-bridge equipped with uprights and gates for just such an emergency, and although it had never been used it was successfully swung over the canal and the uprights and gates were lowered slowly into place. The operation occupied about a day, and two days later the leakage had been stopped sufficiently to permit the commencement of repairs. As the lock was on a rock foundation and there were no banks below to be washed away, no damage of any moment resulted to the structures beyond the tearing out of an upper miter-sill when struck by one of the steamboats, and the spalling of some of the adjacent stone work.

When such an accident happens on a river, recourse must be had to the tem-

porary cofferdam at the head of the lock in order to stop the flow. If this cofferdam is not suited for such a purpose, it may become necessary to sink timber cribs, holding them in place with lines and using only enough stone to secure their sinking, and completing the filling after they are in place. Planks and gravel are then used to make them watertight. In difficult cases the closure may have

Fig. 244.

to be attempted (when the river-bed is not rock) by the method of sandbags and piles described on p. 275 for stopping a crevasse. In one instance when the lock gates had been torn out owing to the collapse of the river lock wall (see p. 674) the flow was stopped by procuring long tree-trunks and forcing them down across the heads of the lock walls into the current. The operation required several

days and was not an easy one. The chamber was 36 feet wide and the foundations were rock.

An instance of breakage due to the age of the gates has been described on p. 481.

The Colorado River and the Salton Basin. In 1905 the Colorado River, one of the largest streams of the United States, abandoned its course not far from the

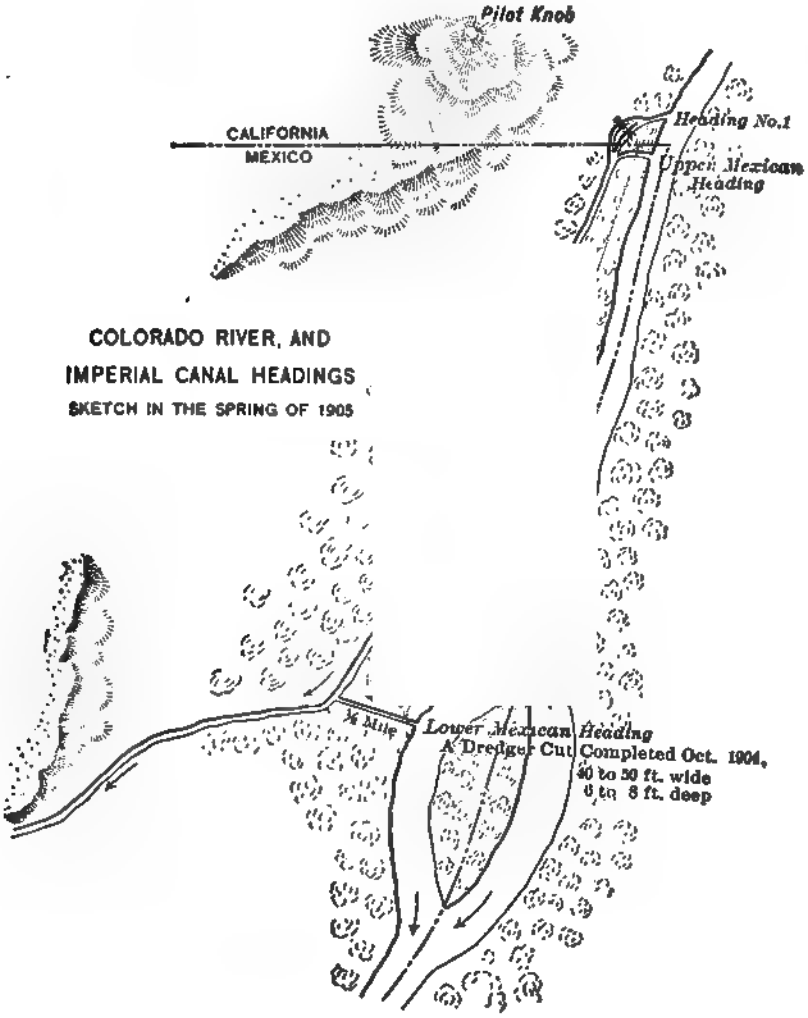


Fig. 245.

mouth, and for more than two years flowed into a vast natural depression known as the Salton Basin, the river-bed below the break being entirely abandoned. The disaster and the means used in remedying it appear to have been unique in the annals of engineering.*

* The accompanying cuts are reproduced by courtesy of Mr. C. E. Grunsky, from his paper in the Transactions, Am. Soc. C.E., 1907. See also the Transactions for 1913 for a later summary by Mr. H. T. Cory.

The Salton Basin lies not far from the mouth of the river, and comprises an area with an extreme length of about 100 miles and an extreme breadth of about 35 miles, all of which is below sea level. The lowest point is about 280 feet below the sea. (Fig. 244.) This area in prehistoric times was a fresh-water lake, and the Colorado River is believed to have flowed into it at practically tide level. By the deposit of sediment, however, the river gradually became cut off from the lake and was diverted towards the sea, filling in the bay at the head of the Gulf of California and creating a "delta-fan" which has built up until the river bed

COLORADO RIVER,
AT LOWER MEXICAN HEADING
OF THE IMPERIAL CANAL.
SKETCH, JUNE 23d, 1905.

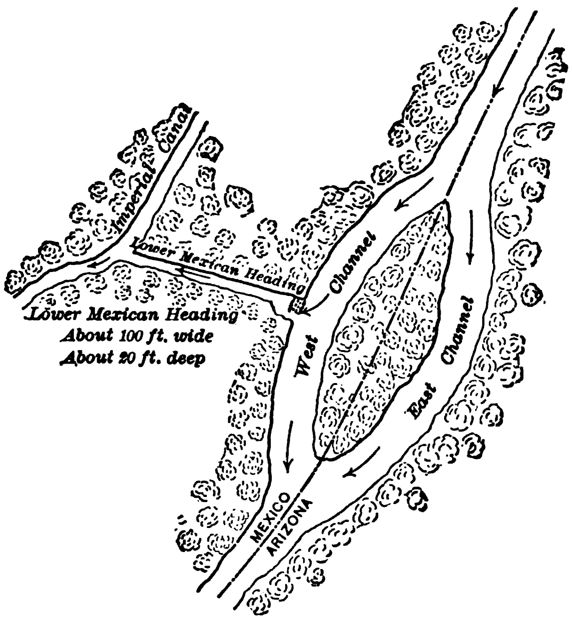


FIG. 246.

at Yuma is now about 120 feet above its ancient level. The lake gradually dried up, leaving extensive deposits of salt in the lowest portion of the depression.

The delta lands are of remarkable fertility, but the climate is hot and very dry, and no crops can be grown without irrigation. A system of canals was accordingly commenced and part of it put into operation in 1901, the water being taken from the Colorado River. Much trouble was occasioned by deposits of silt at the junction of the canal and the river, and during a low-water season in 1904 two new connections were hastily made in order to secure a continuance of the water supply and prevent a total loss of crops (Figs. 245 and 246). No head gates or protections were provided, as it was intended to put them in later. The lower

connection, however, which is said have had an initial fall of $1\frac{1}{2}$ feet in 1300 feet, began to scour rapidly (the soil being of the finest delta silt), and on the advent of a flood quickly got beyond control. Various efforts were made to close it with piles, brush, etc., but the flow merely cut a wider channel and continued the enlargement (Fig. 247), and later attempts to turn the river into its channel on the opposite side of the island also proved futile. By June, 1905, or about nine months after water had been first admitted into the cut, the scour had made a channel more than 100 feet in width and 20 feet in depth, and this was daily increasing. By the time the floods of the ensuing November arrived, the river had abandoned

COLORADO RIVER,
AT THE LOWER MEXICAN HEADING
OF THE IMPERIAL CANAL.
SKETCH SHOWING CONDITION ON MARCH 19TH 1906

Fig. 247.

its old channel and the entire flow was passing into the Salton Basin and recreating the Salton Sea. This condition of affairs continued until November, 1906, every effort to remedy it being foiled by unexpected floods. Twice about 40 miles of the main line of the Southern Pacific Railroad along the Salton Sea had to be moved to higher ground. The river gradually cut ravines several hundred feet in width and from 40 to 80 feet in depth across the alluvial lands, a picture of one of which is shown by the accompanying cuts. These ravines commenced where a steep fall enabled the river to cut out a deep channel, forming a "lip" or cataract, which thereafter traveled rapidly upstream, in some cases at a rate of nearly a mile a day. Near the town of Imperial, early in 1906, the river was flowing in a shallow

The "Barranca" or Ravine Cut by New River, about 4½ Miles Northeast of Brawley, August 30, 1906.
(Cable ferry in mid-ground.)

New River, about 5 Miles Northwest of Imperial, August 31, 1906, Showing the General Character of the Erosion and the new River Bed. (See Fig. 244.)

The Alamo River at Holtville, August 31, 1906, Showing Trestle Built to Carry the Railroad Across the new River Bed. (See Fig. 244.)

Sunset Across the Salton Sea, August 29, 1906, Showing the Lake Recreated by the Colorado River.

The Dry Bed of the Colorado River, View Taken below the Lower Mexican Heading, August 26, 1906. (See Fig. 247.)

View of Dam Built to Close the By-Pass at Lower Mexican Heading, Imperial Canal. (See Fig. 248.) Completed November 13, 1906.

depression, but by August a chasm had been cut there to a depth of 80 feet and with a width of nearly 1200 feet. It was estimated that some 500,000,000 cubic yards of soil were carried from these new channels into the Salton Sea.

In August, 1906, a final effort was made to close the new channel at its inlet from the Colorado River and to turn back the stream into its original channel. A new intake had been excavated at the point where the river left its old bed and wooden regulator gates had been built to control the flow and to pass water for irrigation when the river was to be shut off (see Fig. 247). A railroad trestle was built across the main inlet, then about 600 feet in width, and a series of

LOWER COLORADO RIVER

SKETCH ILLUSTRATING LOCATION OF DAM WHICH TURNED THE RIVER INTO ITS PROPER CHANNEL, NOV. 4TH, 1906.

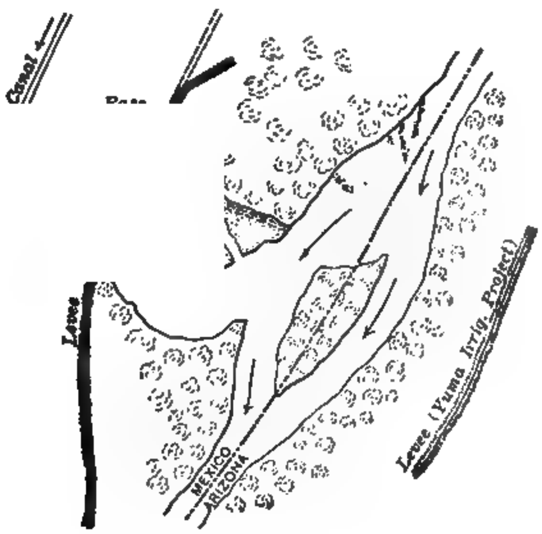


Fig. 248.

brush mattresses weighted with rock were laid as a sill. Upon these large and small rock was thrown as fast as it could be supplied. On October 11, however, when the work was progressing favorably, part of the gates gave way and floated off. causing a new enlargement and making conditions worse than before. Preparations were then made for closing the intake channel by another rock dam and by October 27, this had been successfully accomplished, clay being used on the river face to make it watertight. The dam across the adjoining arm was then continued and on November 4 the river began to flow once more in its original channel (Fig. 248). The final closure was made about three weeks later.

To prevent floods from flanking the dam levees had been built running up- and downstream. These were hastily constructed, without sufficient cut-off trenches,

and made chiefly of material taken from continuous borrow pits on the side away from the river. Late in November a high flood arrived which raised the water at a point a short distance below the dam until it stood about 3 feet deep against the levee. The ground under the levee was full of cracks from former exposure to the sun, and a copious percolation began. The seepage collected in the borrow pit and finally began to flow along it in a volume sufficient to undermine part of the levee. On December 4 a break occurred which soon had a width of 800 feet, and shortly afterwards another break, 500 feet in width, took place, and in a few days the river was again flowing towards the Salton Sea (Fig. 249).

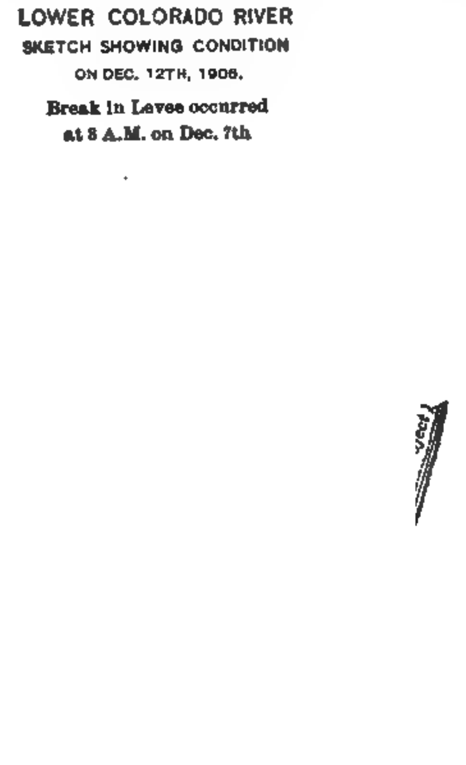


FIG. 249.

Preparations were begun without delay for building a new rock dam. No brush was used, but its place was supplied by throwing in plenty of gravel and small riprap. Two lines of railroad trestles were built across the gap (one of them was destroyed four times by floods and drift) and within three weeks from the time of beginning to dump the rock the closure was successfully accomplished, and on February 11, 1907, the river was again flowing towards the Gulf of California. The levees were strengthened during this interval so as to prevent seepage underneath, and the work has since proved satisfactory. The maximum head on the dam at the time of closure was about 13 feet.

In 1910 another diversion of the river took place, the break occurring about 15 miles below the one just described. It was due to high water entering and enlarging an old flood outlet, and eventually the entire flow became diverted and the river bed was left dry as before. The water, after following old abandoned channels, entered Volcano Lake (see Fig. 244) and came back into the main bed through the channel marked Hardy Colorado. As a continued filling of Volcano Lake would have resulted ultimately in a flow northward into the Salton Sea, followed by erosion sufficient to turn the entire Colorado River into it once more, means were taken to close the break near its head. This was done, in a manner similar to that before described, by dumping rock across the channel, and constructing an accompanying system of levees.

TABLES OF LOCKS AND DAMS.

Dimensions, etc., of works on some of the principal rivers and canals in the United States and elsewhere.
Conditions given are for the beginning of the year 1912.

ALLEGHENY RIVER, PENNSYLVANIA.

No.	Location.	Miles from Mouth.	When Completed.	Lock: Available Dimensions. Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
1(a)	Herr's Island, Pa. . .	1.8	1902	55	286.2	7.0	8.0	Conc.	Grv'l	Movable	710 (a)	Conc.	Gravel	$1,680,000
2	Six Mile Island, Pa.	6.9	1908	56	300.0	11.0	10.0	"	Rock	Fixed	1245	Timber	"	for the
3	Springdale, Pa.	16.9	1904	56	289.6	12.0	7.0	"	"	"	885	"	"	three

(a) Dam consists of three openings—one pass 500 feet long closed by Chanoine wickets (operated from a boat), and two bear-trap weirs each 93 feet long. See p. 656.

BIG SANDY RIVER, WEST VIRGINIA AND KENTUCKY (See p. 563 and after).

No.	Location.	Miles from Mouth.	When Completed.	Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
1	Catlettsburg, Ky. . .	0.3	1904	55	158.0	22.5 (b)	6.0	Conc.	Rock	Movable	310	Conc. founda-	Rock	$318,500
2	Buchanan, Ky.	12.9	1905	55	158.0	12.6	6.0	"	"	"	286	tions;	"	279,000
3	Louisa, Ky.	26.6	1896	52	158.0	10.6	6.0	Stone	"	"	282	mova-	"	356,000
1	Gallup (Levisa Fork)	35.5	1911	55	158.0	12.0	6.0	Conc.	"	"	216	ble dams.	"	268,500
1	Saltpeter, W. Va. (Tug Fork)	31.0	1911	55	158.0	11.0	6.0	"	Grv'l	"	200	See note	Gravel	250,000

(b) Extreme lift with low water in the Ohio River. The normal lift will be about 15 feet.

The Big Sandy dams are all of the same type, having passes of needles (operated from a boat) and weirs of steel Chanoine wickets (operated in part from a boat, but chiefly from service bridges). Each dam has one pass and one weir, of the following respective lengths: Catlettsburg, 140 and 160 feet; Buchanan, 140 and 136 feet; Louisa, 130 and 140 feet; Gallup, 110 and 96 feet; Saltpeter, 110 and 80 feet.

BLACK WARRIOR AND TOMBIGBEE RIVERS, ALABAMA.

No.	Location.	Miles from Mouth.	When Completed.	Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
1	McGrew Shoal, Ala.	111.0	1909	52	286.0	10.4	6.6	Conc.	Shale Clay	Fixed	500	Conc.	Shale Clay	
2	Clear Creek.	182.0	Incomplete	52	286.0	10.0	6.5	"	Clay	"	500	Timber	Clay	
3	Griffin Landing	204.9	"	52	286.0	10.0	6.5	"	"	"	500	"	"	
4	Demopolis.	230.6	1908	52	286.0	10.0	6.5	"	"	"	479	"	"	Cost of
5	Candy's Landing. . .	246.2	1908	52	286.0	10.0	6.5	"	"	"	299	"	"	complet-
6	Erie Bar.	266.7	1908	52	286.0	10.0	6.5	"	Gravl	"	299	"	Gravel	ed works
7	Pickens Bar	282.3	1903	52	286.0	10.0	6.5	"	"	"	300	"	"	$4,020,000
8	Log Shoals.	298.3	1903	52	286.0	10.0	6.5	"	"	"	340	"	"	of incom-
9	Mike's Bar.	315.2	1902	52	286.0	10.0	6.5	"	"	"	300	"	"	plete
10	Tuscaloosa.	361.9	1895	52	286.0	9.9	6.6	Stone	Rock	"	339	"	"	works,
11	"	362.2	1895	52	286.0	8.5	6.5	"	"	"	409	"	Rock	(esti-
12	"	363.0	1895	52	286.0	10.5	6.5	"	"	"	650	"	"	mated)
13	Tidewater.	370.1	1905	52	286.0	12.1	6.5	"	"	"	640	Stone	"	$1,500,000
14	Arnold's Shoals. . . .	373.5	1910	52	286.0	14.0	6.5	Conc.	"	"	930	Conc.	"	Total,
15	Rose Shoals.	380.2	1910	52	286.0	14.0	6.5	"	"	"	870	"	"	$5,520,000
16	Squaw Shoals.	386.5	Incomplete	52	286.0	21.0	6.5	"	"	"	1045	"	"	
17(c)	. . . .	. . . . .	Projt'd	. . . . .	. . . . .	. . . .	. . . .	. . . . .	. . . . .	. . . . .	(c)			
18(c)	. . . .	. . . . .	"	. . . . .	. . . . .	. . . .	. . . .	. . . . .	. . . . .	. . . . .	(c)			

(c) Projected with locks 17 and 18 as a flight, and one dam with a lift of 63 feet.

NOTE: For locations of rivers see Pl. 46.

BRAZOS RIVER, TEXAS.

No.	Location.	Miles from Mouth.	When Completed.	Lock: Available Dimensions. Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
....	Hidalgo Falls, Tex.	.....	1911	55	140	11.0	6.0	Conc.	Rock	Movable	.....	See note	Rock	....

The dam has one pass of Chanoine wickets, one weir of Boulé gates, and one bear-trap. Others are under construction.

COLUMBIA RIVER, OREGON.

.....	Cascades, Oregon...	155.0	1896	90	475	24.0 max.	8.0	Conc. & St.	Rock	Dams are formed by natural obstructions				....
.....	Big Eddy.........	200.0	Incomplete	45(*d*)	273	70.0	7.0	Conc.	"					
.....	Five Mile Rapids...	201.7	"	45	273	18.5	7.0	"	"					
.....	Ten Mile Rapids...	205.5	Proj'd	45	273	5.0	7.0	"	"					
.....	Celilo Falls........	208.0	Incomplete	45	273	14.0	7.0	"	"					

(*d*) Tandem lock with a total lift of 70 feet.

CONGAREE RIVER, SOUTH CAROLINA.

.....	Below Columbia, S.C	.....	1903	55	170	8.0	6.0	Conc.	Rock	Movable	376	Conc. base	Rock	$250,000 (estd.)

Dam consists of a single opening, closed by Chanoine wickets operated from a trestle bridge.

COOSA RIVER, GEORGIA AND ALABAMA.

1	....	.....	......	40	189	5.3	....	Stone	.....	Fixed	2000	Timber	......	....
2	....	.....	......	40	189	5.6	....	"	.....	"	1290	"	......	....
3	....	.....	......	40	189	12.0	....	"	.....	"	180	"	......	....
.....	Mayo's Bar...........	.....	Incomplete	40	189	12.0	....	Conc.	.....	"	325	"	......	....
4	....	.....	"	52	322	9.0	....	"	.....	"	700	"	......	....

CUMBERLAND RIVER, TENNESSEE AND KENTUCKY.

C	Sailor's Rest, Tenn..	109.1	Incomplete	52	300	12.0	6.6	Conc.	Rock	Fixed	360	Conc.	Rock	
B	Hickory Pt., Tenn. .	140.9	"	52	300	12.0	6.6	"	"	"	440	"	"	
A	Fox Bluff, Tenn....	151.2	1904	52	300	12.75	6.5	Stone	"	"	535	Timber	"	
1	Nashville, Tenn....	190.0	1904	52	300	6.5	6.5	"	"	"	439	"	"	$2,907,000 for completed works
2	Madison, Tenn.....	201.6	1907	52	300	11.0	6.5	"	"	"	411	"	"	
3	Hendersonville, Tenn	228.6	1908	52	300	12.0	6.5	"	"	"	410	"	"	
4	Gallatin, Tenn.....	237.3	1909	52	300	12.0	6.5	"	"	"	455	"	"	
5	Lebanon, Tenn.....	264.8	1909	52	300	12.0	6.5	"	"	"	505	"	"	
6	Hartsville, Tenn....	282.2	1910	52	300	12.0	6.5	"	"	"	400	"	"	
7	Carthage, Tenn....	299.2	1910	52	300	12.0	6.5	"	"	"	416	"	"	
21	Burnside, Ky......	486.8	1911	52	300	14.0	6.5	Conc.	"	"	340	Conc.	"	

NOTE: For locations of rivers see Pl. 46.

FOX RIVER, WISCONSIN.

No.	Location.	Miles from Mouth.	When Completed.	Lock: Available Dimensions. Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length, Feet.	Built of	Built on
1	Depere, Wis.......	0	1897	35.8	170.0	9.0	7.0	Most of the locks are of cut stone; a few are of stone and timber, as described on p. 384.	Rock	Fixed	987.8	Timber	Nos. 1 to 18 are on rock
2	Little Kaukauna ..	5.9	1896	36.6	160.5	7.1	6.7		Earth	"	588.7	Piles	
3	Rapide Croche....	11.9	1859	36.6	160.4	8.0	7.7		"	"	578.0	Timber	
4	Kaukauna, Fifth. .	15.7	1898	35.6	170.0	8.6	8.9		Rock		No dam	....	
5	" , Fourth	16.0	1878	36.6	170.1	10.0	6.0		"		"	....	
6	" , Third .	16.2	1878	36.6	170.0	10.4	6.0		"		"	....	
7	" , Second	16.4	1903	35.0	170.0	10.2	6.2		"		"	....	
8	" , First...	16.6	1883	35.0	170.4	9.9	6.0		"	"	582.0	Timber	
9	Little Chute, Lower	18.3	1878	36.4	172.5	11.0	8.1		"		No dam	....	
10	" Upper.	18.3	1878	36.3	170.1	10.8	6.0		Hardpan		"	....	
11	" Second	17.3	1881	35.0	170.2	14.5	6.0		Rock		"	....	
12	" First..	17.5	1894	35.4	160.2	.....	11.6		"	"	690.0	Timber	
13	Cedars..........	20.3	1887	35.0	170.0	9.8	6.8		"	"	820.0	"	
14	Appleton, Fourth. .	23.7	1895	35.0	160.2	8.2	7.7		"	"	540.0	"	
15	" , Third. ..	24.2	1900	35.0	170.0	8.5	8.7		"		No dam	....	
16	" , Second. .	24.5	1901	34.8	170.6	9.7	6.0		Clay		"	....	
17	" , First....	24.9	1884	35.0	170.7	10.1	6.0		Rock	"	800.0	Stone	
18	Menasha........	30.2	1899	35.4	170.0	9.2	6.2		Clay	"	401.7	Timber	
19	Eureka..........	73.8	1877	35.7	170.6	2.7	10.2		Piles	"	209.0	"	Nos. 19 and above are on foundations varying from rock to sand
20	Berlin...........	82.1	1878	34.8	170.6	2.1	8.9		Clay	"	200.0	"	
21	White River......	92.1	1878	34.5	170.5	1.4	10.9		"	"	200.0	"	
22	Princeton........	101.5	1878	34.9	170.4	0.9	9.9		Sand	"	180.0	"	
23	Grand River......	122.2	1878	34.7	170.3	1.1	10.0		Clay	"	120.0	"	
24	Montello.........	125.5	1901	35.3	160.5	3.1	6.1		Sand	"	176.6	"	
25	Governor Bend....	149.6	1900	35.0	160.4	3.1	5.6		"	"	59.6	"	
26	Ft. Winnebago....	153.8	1901	34.3	160.2	5.9	5.9		Clay	"	60.0	"	
27	Portage..........	156.0	1901	35.1	165.3	9.0	5.1		Sand	......	....	....	

GALENA RIVER, ILLINOIS.

1	Galena Junc., Ill...	215	1894	52	273.5	5.0	.....	Stone	Rock	Fixed	123	Timber	Rock, & mud

GREEN, BARREN, AND ROUGH RIVERS, KENTUCKY.

1	Spottsville........	8.5	1840	36	138	19.1	6.0	Stone	Rock	Fixed	528.5	Timber	Rock
2	Rumsey.........	62.8	1896 (e)	36	145	15.4	5.1	"	"	"	598.0	"	"
3	Rochester........	108.5	1840	36	138	15.4	6.8	"	"	"	353.0	"	"
4	Woodbury........	149.5	1840	36	138	16.7	5.6	"	"	"	381.0	"	"
5	Glenmore.........	167.5	1899	36	145	14.0	5.1	Conc.	Gravel	"	282.0	"	Gravel
6	Below Mammoth Cave..........	180.7	1906	36	145	11.0	6.0	"	"	"	220.0	Piles and conc.	"
1	Barren River.....	165.0	1840	36	140	15.1	4.5	Stone	Rock& gravel	"	268.0	Timber	Rock
1	Rough River.....	79.0	1896	27	123	9.0	5.2	Conc.	Rock	"	185.0	"	"

(e) Rebuilt; originally built about 1840.

NOTE: For locations of rivers see Pl. 46.

HUDSON RIVER, NEW YORK.

(Part of the Champlain Division of the New York State Barge Canal. See also pp. 695 and 696.)

No.	Location.	Miles from Mouth.	When Completed.	Lock: Available Dimensions. Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
1	Troy, N. Y. (Erie Canal number)	—	After 1912	45	483–493 (f)	15.2	16½	Conc.	Rock	Fixed	1275 abt.	Conc.	Rock	For cost of construcsee p. 390. "Cost of Canalization"
1	Waterford, N. Y.	Fr. Troy abt. 5 m.	1912	45	300–310	14.3	12.0	"	"	"	847	"	"	
2	Mechanicsville..	do. 9 m.	1912	45	300–310	18.5	12.0	"	"	"	850	"	"	
3	" ..	do. 11⅓m	1912	45	300–310	19.5	12.0	"	"	"	690	Stone &conc.	"	
4	Stillwater......	do.13⅓m.	1911	45	300–310	16.0	12.0	"	"	"	818	Conc.	"	
5	Northumberland	...27⅓m.	1911	45	300–310	19.0	12.0	"	"	"	835	"	"	
6	Ft. Miller......	do. 31 m.	1909	45	300–310	16.5	12.0	"	"	"	760	"	"	

(f) Divided into two chambers by intermediate gates, the longer chamber being 300 to 310 feet in available length, depending upon the shape of the bow of the boat.

ILLINOIS RIVER, ILLINOIS.

..	Kampsville, Ill. .	29.5	1893	73	324.5	6.13	9.5	Cut stone	......	.....	1200	Piles & stone	Earth	....
..	La Grange......	75.0	1889	73	324.5	5.65	9.7	"	......	.....	819	"	"	....

KANAWHA RIVER, WEST VIRGINIA.

(See p. 595.)

No.	Location.	Miles from mouth.	Completed.	Lift, Feet.	Lock. Built of	Built on	Dam. Total Gross Length.	Weir. Composed of	Length, Feet.	Pass. Composed of	Length, Feet.	Cost of Works.
2	Montgomery, W. Va. ..	85.2	1887	10.3	Stone	Rock	524	} Fixed	da	ms	{	$353,600
3	Paint Creek..........	80.0	1882	13.7	"	"	564.5					490,000
4	Coalburg............	73.7	1880	7.2	"	"	468	Ch. wkts.	210	Ch. wkts.	248	275,000
5	Marmet............	67.7	1880	7.5	"	"	529	"	265	"	250	275,000
6	Charleston (4 miles distant)..............	54.5	1886	8.5	"	"	568	"	310	"	248	337,600
7	St. Albans...........	44.2	1893	8.2	"	"	574	"	316	"	248	341,200
8	Winfield............	36.0	1893	8.0	"	"	550	"	292	"	248	281,900
9	Fraziers Bottom.......	25.2	1898	6.2	"	"	542	"	284	"	248	315,000
10	Buffalo.............	18.7	1898	7.0	"	"	542	"	284	"	248	290,000
11	Point Pleasant........	1.7	1808	10.	"	Hard-pan	678	"	364	"	304	650,000

The chambers of Locks 2, 3, 4, and 5 are 50 feet in width and 271 to 274 feet in available length. The remaining locks are 55 feet in width and 313 feet in available length. The least channel depth is 6 feet. All the movable dams are operated from service bridges.

NOTE: For locations of rivers see Pl. 46.

KENTUCKY RIVER, KENTUCKY.

No.	Location.	Miles from Mouth.	Completed.	Lock: Available Dimensions. Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length.	Built of	Built on	Cost of Works.
1	Carrollton, Ky..	4.0	1844	38	151	17.0	6.0	Ct.st.	Rock	Fixed	423.7	Tim.&conc.	Clay	
2	Lockport.......	31.0	1844	38	151	13.9	6.0	"	"	"	400.0	"	Clay & rock	
3	Gest...........	42.0	1844	38	151	12.5	6.5	"	"	"	465.0	"	Rock	Cost of
4	Frankfort......	65.0	1844	38	151	13.9	6.0	"	"	"	543.0	"	"	complet-
5	Tyrone........	82.2	1844	38	151	14.5	6.7	"	"	"	556.0	"	"	ed works,
6	Warwick.......	96.2	1891	52	153	14.0	6.0	"	"	"	413.0	"	Clay & rock	Nos, 1 to
7	High Bridge....	117.0	1897	52	153	15.3	6.0	"	"	"	350.0	Timber	Rock	12 incl.,
8	Little Hickman..	139.2	1900	52	153	18.0	6.0	"	"	"	257.0	Tim.&conc.	"	$2,940,000
9	Valley View.....	157.0	1903	52	153	18.0	6.0	Conc.	"	"	363.0	Concrete	"	Do. of
10	Ford..........	176.5	1904	52	153	17.0	6.0	"	"	"	466.0	"	"	Nos. 13
11	College Hill.....	200.0	1906	52	153	18.0	6.0	"	"	"	208.0(g)	"	"	and 14
12	Irvine.........	220.5	1910	52	153	18.0	7.0	"	"	"	248.0(g)	"	"	(estd.)
13	Thirteen.......	239.5	Incm.	52	153	18.0	7.0	"	"	"	240.0(g)	"	"	$772,000.
14	Heidelberg.....	248.5	"	52	153	18.0	7.0	"	"	"	240.0(g)	"	"	Total:
														$3,712,000

(g) These four dams have movable crests of needles. See p. 521 and after.

LITTLE KANAWHA RIVER, WEST VIRGINIA.

1	Shacktw'n, W.Va.	3.5	1909*	22	125	15.7	3.5	Ct.st. & con.	Rock	Fixed	281	Timber	Rock	Nos 1 to 4 inc. were
2	Leaches........	14.0	1907*	22	125	10.2	3.5		"	"	274	"	"	bought
3	Wells..........	26.2	1908*	22	125	11.8	3.5	"	"	"	289	"	"	by the
4	Palestine.......	32.0	1906*	22	125	12.0	3.5	"	"	"	282	"	"	U. S. in
5	Burning Springs.	40.2	1891	26	126	12.0	4.0	Ct.st.	Piles in Gr.	"	250	"	"	1905 for $75,000

* Rebuilt in part; originally built in 1867 by a private corporation.

MISSISSIPPI RIVER.

1	Minneapolis, Minn........	1942.0	Incm.	80	315	31.5	9	Conc.	Rock	Fixed	575	Concrete	Gravel	
2	St. Paul, Minn. .	1945.8	1902	80	310	13.0	5	"	"	"	521	Timber	Rock	
	Keokuk(h), Ia...	1444.0	1877	79	296	11.7	6	Ct.st.	"	} Dam formed by natural obstructions				
	Keokuk(h), Ia...	1446.5	1877	79	296	8.0	5	"	"					
	Keokuk(h), Ia...	1451.5	1877	79	296	6.0	5	"	"					
	Keokuk (power dam with lock)	......	Incm.	110	400	40.0	8	Conc.	Rock	(j)	4278	Concrete	Rock	
	Moline, Ill.....	1567.0	1907	80	294	7.5	14	Conc.	Rock	.....	Dam formed by natural obstructions			

(h) Locks of lateral canal; to be replaced by the single lock described just below.
(j) Bridge dam; see p. 637.

NOTE: For locations of rivers see Pl. 46.

MOHAWK RIVER, NEW YORK (Erie Division of the New York State Barge Canal).

(See pp. 632 and after and pp. 695 and 696.)

No.	Location.	When Completed.	Lock: Available Dimensions. Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length, Feet.	Built of	Built on
1	Troy, N. Y.	(See Hudson River)										
2	Waterford.........	The locks and dams were constructed between 1905 and 1912, and completed by 1912 with the exception of Scotia and the dams at Crescent and Visscher's Ferry.	45	300–310	34.5	12.0	Conc.	Rock	No dam *			
3	“		45	300–310	34.5	12.0	“	“	“			
4	“		45	300–310	34.5	12.0	“	“	“			
5	“		45	300–310	33.25	12.0	“	“	“			
6	Crescent..........		45	300–310	33.0	12.0	“	“	Fixed *	1486.2	Conc.	Rock
7	Visscher's Ferry....		45	300–310	27.0	12.0	“	“	“	1918.7	“	“
8	Scotia............		45	300–310	14.0	12.0	“	Gravel	Movable (Bridge & gates).	530	“	Gravel
9	Rotterdam.........		45	300–310	15.0	12.0	“	Sandy clay.	“	530	“	Sandy clay
10	Crane's Village.....		45	300–310	15.0	12.0	“	Gravel	“	500	“	S. & mid-spans, gravel; N. span, hardpan
11	Amsterdam........		45	300–310	12.0	12.0	“	Rock	“	590	“	Gravel
12	Tribe's Hill........		45	300–310	11.0	12.0	“	“	“	492	“	S.½ sandy clay, N½ rock.
13	Yosts............		45	300–310	8.0	12.0	“	Gravel	“	370	“	Gravel
14	Canajoharie.......		45	300–310	8.0	12.0	“	Rock	“	430	“	“
15	Fort Plain.........		45	300–310	8.0	12.0	“	“	“	430	“	Rock
16	Mindenville.......		45	300–310	20.5	12.0	“	“	Fixed, movable, Boulé crest.	360.2		“
17	Little Falls........		45	300–310	40.5	12.0	“	“	Fixed	....	“	“
18	Jacksonburg.......		45	300–310	20.0	12.0	“	“	Movable (needles)	126	“	Gravel

* The last navigation dam in the Mohawk is at Crescent; the waterway then passes down to the Hudson River across the hills through Locks 2, 3, 4, 5, and 6.

Total distance from Lock No. 2 to Lock No. 18, about 94½ miles; total fall, about 368 feet. For cost see pp. 391 and 639.

The dams corresponding to Locks 8 to 15 inclusive are of the bridge type (see p. 632) and of the following widths of net openings, measured at the pool level: At Scotia, 2 of 150 and 210 feet; at Rotterdam the same; at Cranes Village, 2 of 150 and 1 of 180 feet; at Amsterdam, 2 of 180 and 1 of 210 feet; at Tribes Hill, 2 of 240 feet; at Yosts, 2 of 180 feet; at Canajoharie, 2 of 210 feet; and at Fort Plain, 2 of 210 feet. The dam at Scotia has a depth from sill to upper pool of 19 feet; those from Rotterdam to Amsterdam inclusive have 20 feet; those at Yosts and Fort Plain have 18 feet; and those at Tribes Hill and Canajoharie have 16 feet. The dam for the Mindenville lock is situated at Rocky Rift, about 4 miles upstream, and is referred to on p. 522. The dam for the Jacksonburg lock is at Herkimer, about 4½ miles upstream and is referred to on p. 561.

The available length of the lock depends upon the shape of the bow of the boat.

NOTE: For locations of rivers see Pl. 46.

MONONGAHELA RIVER, PENNSYLVANIA AND WEST VIRGINIA

No.	Location.	Miles from Mouth.	When Completed.	Lock: Available Dimensions. Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
1	Pittsburgh, Pa....	1.9	1909†	56*	360.0	6.6	10.0	Conc.	Rock	Fixed	962.5	Tm.-Cn	Gravel	Cost, including $2,210,000 paid by the U. S. in 1896 for locks and dms. 1 to 7 incl. (till then owned by a private corporation), $5,632,000
2	Braddock, Pa.....	11.2	1906†	56*	362.5	7.8	8.7	"	Gravel	"	808.2(*k*)	Conc.	"	
3	Elizabeth, Pa.....	23.8	1907†	56*	360.0	8.6	8.5	"	Rock	"	685.3(*k*)	"	"	
4	N. Charleroi, Pa...	41.2	1844	56*	227.0	11.1	5.8	Cut stn.	Gravel	"	703.0	Timber	"	
5	Brownsville, Pa...	56.7	1910†	56*	360.0	11.8	5.5	Conc.	Rock	"	553.3(*k*)	Conc.	"	
6	Rice's Landing, Pa.	68.7	1856	50	165.0	12.8	6.2	Cut stn.	"	"	626.0	Timber	"	
7	Geneva (2 miles above), Pa......	82.7	1883	50	159.0	9.4	6.5	"	"	"	520.0	"	"	
8	Dunkard Creek, Pa.	87.5	1889	50	161.0	10.6	5.0	"	"	"	587.5	"	"	
9	Hoard, W. Va....	93.3	1879	50	160.0	12.2	5.6	"	"	"	388.7	Cut stn.	Rock	
10	Morgantown, W. Va	102.6	1903	56	182.0	11.2	7.0	Conc.	"	"	444.3	Conc.	"	
11	Uppington, W. Va.	104.9	1903	56	182.0	10.7	7.0	"	"	"	500.0	"	"	
12	Little Falls (below) W. Va.........	109.8	1904	56	182.0	10.7	7.0	"	"	"	425.0	"	"	
13	Do. (above)......	111.8	1904	56	182.0	10.7	7.0	"	"	"	410.0	"	"	
14	Lowsville, W. Va...	115.5	1904	56	182.0	10.7	7.0	"	"	"	446.0	"	"	
15	Hoult, W. Va.....	124.1	1904	56	182.0	10.7	7.0	"	"	"	430.0	"	"	

(*k*) Dam has a movable crest; see p. 522.

MUSKINGUM RIVER, OHIO.

No.	Location.	Miles from Mouth.	When Completed.	Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
1	Marietta, O......	0.2	1890	56	360.0	12.8	6.0	Cut stn.	Piles	Fixed	480	Timber	Gravel	Cost: $1,857,000 Nos. 2 to 10 incl. were originally built by the State of Ohio at a cost of $1,300,000 and were transferred to the U. S. free of charge in 1887
2	Devols...........	5.7	1840	36	162.7	10.5	5.4	"	Rock	"	588	"	Rock	
3	Lowell...........	13.9	1891†	36	162.7	14.5	5.5	"	Piles	"	848	"	"	
4	Beverly..........	24.7	1840	36	162.7	9.4	4.5	"	Gravel	"	705	Tm.-cn.	Rock& gravel	
5	Luke Chute......	33.0	1840	36	162.7	10.9	5.5	"	Rock	"	546	"	"	
6	Stockport........	39.1	1891†	36	162.7	12.0	5.5	"	"	"	482	Timber	Rock	
7	McConnelsville....	48.2	1891†	36	162.7	10.1	5.5	"	"	"	472	Tm.-cn.	Rock& gravel	
8	Eagleport........	56.0	1891†	36	162.7	11.0	5.5	"	Piles	"	515	"	Gravel	
9	Taylorsville......	66.8	1889†	36	162.7	11.3	5.5	"	Rock	"	745	"	Rock	
10	Zanesville........	75.8	1840	36*	159.0	15.7	7.0	"	Rock & gravel	"	514	Timber	"	
11	Ellis.............	83.9	1910	36	160.7	11.4	7.1	Conc.	Piles	"	340 (*l*)	Conc.	Rock& gravel	

* Double lock. † Rebuilt at date given.

(*l*) Dam has a movable crest of Boulé gates; see p. 522.

NOTE: For locations of rivers see Pl. 46.

NEW YORK STATE BARGE CANAL.

This canal, whose construction was begun in 1905, is one of the largest and most complicated pieces of construction of modern times. It is a modernization of the Erie Canal and its connections, a system which was completed between 1825 and 1830. The new canal commences at Troy, N. Y., at the head of the tidewater navigation of the Hudson River (about 150 miles from the city of New York) and runs almost due west for a distance of about 340 miles, connecting near Buffalo with the outlet of Lake Erie. The Champlain division connects near Troy, and runs due north to the south end of Lake Champlain, from the north end of which lake connection is made with the St. Lawrence by following the Richelieu River and the Chambly canal. The Oswego division connects with the main line not far from Syracuse, N. Y., as does also the Cayuga-Seneca division. The former runs north to Lake Ontario, following the Oswego River; the latter runs westerly, following the Seneca River, and connects with Lakes Cayuga and Seneca.

All the locks are single, 45 feet in width, and from 300 to 310 feet in available length (328 feet between hollow quoins), depending upon the design of the boat, with a least depth on miter-sills of 12 feet, and lifts ranging from 6 to 40½ feet. All lock gates are of steel, and of the straight mitering type, with the exception of the lower gate of the Little Falls lock, which is of the lifting type.

The canal prism has a least bottom width of 75 feet in earth sections, and of 94 feet in rock sections, giving minimum areas of 1128 square feet in each case. Most of the river channels were made 200 feet in width. The minimum depth of water is 12 feet. The canal as originally planned was to have locks 28 feet in width; this was changed later to 45 feet, but the prism was retained as 1128 square feet, and must be increased in the future if it is to suit the capacity of the locks. The least headroom under bridges is 15½ feet.

The main stem, or the Erie division, about 340 miles in length, commences at Troy, N. Y., as stated above, and follows the Hudson River for about two miles, thence turning westerly across the hills into the Mohawk River, and following this stream to within a few miles of Utica, N. Y.* At that point it enters a canal section and reaches near Rome the summit level of 420 feet above mean sea level. This elevation is reached by 20 locks, including the one at Troy. From Rome the canal continues westward, stepping down through Oneida Lake into the Oneida River by means of 3 locks, the total fall being 57 feet. Passing down the Oneida River the junction of the Oswego, Seneca, and Oneida rivers is reached at Three Rivers, at which point the Oswego Canal begins and runs northward to Lake Ontario. The main canal then ascends the Seneca and Clyde rivers, and continues westerly to

* See table of locks, etc., for the Mohawk River, p. 693.

near Rochester, N. Y., where it crosses the Genesee River at pool level and then follows the contour of an ancient sea shore for about 60 miles, finally connecting at Tonawanda with the Niagara River and the Great Lakes. From the Oneida River to Tonawanda the rise is 202.6 feet and is overcome by 12 locks. The total net rise from sea level is 565.6 feet, and the total number of locks is 35. There is in addition a considerable number of fixed and movable dams, several small locks connecting with parts of the old canal system, some guard locks, and a large number of guard gates of the lifting sluice-gate type. The Rome summit level is supplied from some old feeders and from two new storage dams, each of which impounds a lake about $4\frac{1}{3}$ square miles in area.

The Champlain division follows the Hudson River from Troy to Fort Edward,* and then rises to its summit level of 140 feet above mean sea level. This summit is fed from the upper Hudson River and is reached by 8 locks, not including the one at Troy. From the summit a descent of $43\frac{1}{2}$ feet is made to Lake Champlain by means of 3 locks. The total length is about $61\frac{1}{2}$ miles.

The Oswego division leaves the main line near and north of Syracuse, N. Y., and consists of a canalization of the Oswego River, ending at Lake Ontario. The total fall is 118.6 in about 23 miles, and is overcome by 7 locks,† the elevation of the lake being 244.4 above mean sea level.

The Cayuga-Seneca division leaves the main line to the west of Syracuse, N. Y. and ascends the Seneca River, connecting with Lakes Cayuga and Seneca. The total rise from the Erie level is 71 feet, overcome by four locks. The elevation of Seneca Lake is 445.0 feet above mean sea level.

The preliminary estimates of construction gave the total amount of excavation as about 135,000,000 cubic yards, and the total amount of concrete as about 3,250,000 cubic yards. The total estimated cost of all the divisions named was about $108,000,000, a large part of which was for damages to water-powers and other property.

* See table of locks, etc., for the Hudson River, p. 691.

† See table of locks, etc., for the Oswego River, p. 698.

OHIO RIVER.

(Condition in 1912; see pp. 591 and 595 and after. The system is to be extended to near Cairo at the mouth of the river.)

No.	Location.	Miles from Mouth.	When Completed.	Lock.		Foundations.	Dam.			Cost of Lock and Dam (See notes)
				Lift, Feet.	Built of		Total Length of Dam, Ft.	Weir, Number of parts, type, and length.	Length of Pass, Ft.	
1	West Bellevue (Davis Is.), Pa.	962.3	1885	3.13	Stone	Rock & gravel	1223	2 Ch. wkt. 212′, 216′, 1 bear-trap 52′	559	$ 973,700
2	Coraopolis, Pa. .	958.0	1906	7.74	Cn.-st.	Rock	1002	1 " 176′, 1 " 102′	700	977,200
3	Glenosborne, Pa.	956.1	1908	7.74	"	Gravel	1068	1 " 144′, 2 bear-traps 93′ each....	700	1,157,200
4	Legionville, Pa. .	948.4	1908	7.64	"	"	1108	1 " 184′, 2 " 93′ "	700	1,085,900
5	Freedom, Pa....	943.1	1907	8.48	"	"	1020	1 " 96′, 2 " 93′ "	700	1,086,000
6	Vanport, Pa....	938.2	1904	5.67	Stone	Rock & gravel	1000	1 A-frame 120′, 2 " 120′ "	600	1,123,700
7	Midland, Pa....	930.0	Incom.	6.9	Conc.	Rock	892	1 Ch. wkt. 72′, 2 " 91′ "	600	1,155,400
8	Kenilworth, W. Va.	921.0	1911	6.4	"	Gravel	1120	1 " 200′, 2 " 91′ "	700	1,166,300
9	New Cumberland, W. Va...	911.4	Incom.	7.4	"	"	1044	1 " 224′, 2 " 91′ "	600	1,345,700
10	Steubenville, O. .	903.5	"	8.4	"	Rock	1076	1 " 256′, 2 " 91′ "	600	1,300,400
11	Wellsburg, W. Va.	890.7	1910	7.3	"	Gravel	1040	1 " 120′, 2 " 91′ "	700	1,010,000
12	Wheeling, "	880.0	Incom.	8.4	"	"	1120	1 " 200′, 2 " 91′ "	700	1,365,300
13	McMechen, "	871.2	1909	7.3	"	"	996	(No wicket weir), 2 bear-traps 83′, ea. (†)	800	1,150,000
14	Woodland, "	853.0	Incom.	8.3	"	Rock	956	1 Ch. wkt. 136′, 2 bear-traps 91′ each....	600	1,284,200
15	N. Martinsville, "	838.0	"	7.8	"	"	1028	1 " 208′, 2 " 91′ "	600	1,143,100
16	Ben's Run, "	821.0	*	7.8	"	"	980	1 " 160′, 2 " 91′ "	600	1,302,700
17	Marietta (4 mi. above), O....	800.0	*	8.2	"	"	1272	1 " 352′, 2 " 91′ "	700	1,311,300
18	Do. (8 m. below) "	787.7	1909	6.2	"	"	1135	1 " 300′, 2 " 50′, "	700	910,000
19	Little Hocking, "	775.6	Incom.	7.7	"	Rock & gravel	1224	1 " 304′, 2 " 91′ "	700	1,150,000
20	Bellville, W. Va.	766.0	"	7.5	"	Rock	1012	1 " 192′, 2 " 91′ "	600	1,041,700
21	Portland, O.....	753.0	*	5.6	"	"	1116	1 " 296′, 2 " 91′ "	600	1,111,250
22	Ravenswood, W. Va.	747.0	*	7.8	"	"	1044	1 " 224′, 2 " 91′ "	600	1,069,700
23	Millwood, "	737.0	*	8.1	"	"	1012	1 " 192′, 2 " 91′ "	600	1,024,000
24	Graham, "	726.0	*	7.0	"	"	1028	1 " 208′, 2 " 91′ "	600	986,200
25	Pt. Pleasant, "	707.0	*	9.0	"	"	964	1 " 144′, 2 " 91′ "	600	1,076,100
26	Hogsett, "	693.0	1912	7.5	"	"	1092	1 " 272′, 2 " 91′ "	600	1,051,900
27	Proctorsville (4 mi. above), O.	679.0	*	6.4	"	"	1160	1 " 240′, 2 " 91′ "	700	1,041,300
28	Huntington (2 m. below), W. Va.	668.0	Incom.	7.1	"	"	1216	1 " 296′, 2 " 91′ "	700	883,000
29	Ashland (2 mi. above), Ky.	647.0	"	8.0	"	"	1044	1 " 224′, 2 " 91′ "	600	847,600
37	Fernbank, O. ...	485.7	1911	7.8	"	"	1184	(No wicket weir), 3 bear-traps 80′ each.	900	1,300,000
41	Louisville (see notes), Ky.	363.0	Incom.	8.0	"	"	5046	648′ of Ch. wickets; 1952′ of Boulé gates; 2446′ of masonry fixed weir..	None ‡	1,854,000
48	Henderson (6 mi. below), Ky.	163.0	"	11.0	"	Sand.	2290	1 Ch. wicket of 200′ and 1 of 400′, and 890′ of masonry fixed weir.........	800	1,700,000

NOTES.—All locks are 110′×600′ in available dimensions; navigable depth of channel, 9′ 0″; lock gates are generally of the rolling type of steel. All Passes are closed by Chanoine wickets, of wood, operated from boats; weirs are also closed by wickets (except at Nos. 13 and 37) but operated from bridges. Bear-trap drift-chutes are also provided.

Lock No. 41 at the Louisville and Portland Canal is to be rebuilt and the canal is to be enlarged. Costs given for Nos. 1 to 6, and for Nos. 8 and 37, are actual: others are approximate.

* Not commenced, June, 1912.

(†) Dam No. 13 bear-traps are of the three-leaved type.

‡ The dam is at the head of the Falls of the Ohio. Boats can pass these only on a good stage of water.

NOTE: For locations of rivers see Pl. 46.

OSAGE RIVER, MISSOURI.

No.	Location.	Miles from Mouth.	When Completed.	Lock: Available Dimensions. Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
1	Osage, Mo.	7.0	1912	42	196	16.0 max.	7.0	Conc.	.....	Movable	840	Conc.	Sand & gravel	$736,000

The dam consists of a pass 415 feet long, closed by Chanoine wickets, and a weir of 375 feet net length, closed by five Chittenden drum wickets, each 75 feet long and separated by piers 10 feet wide. (See p. 649.)

OSWEGO RIVER, NEW YORK.

(Canalized for the New York State Barge Canal. See pp. 395 and 396.)

1	Phœnix, N. Y.	21.3	1912	45	300 to 310	10.2	12.0	Conc.	Rock	Fixed with sluice gates	206.0	Conc.	Rock	....
2	Fulton	12.0	After 1912	45	"	17.8	12.0	"	"	Fixed	406.9	Cut-stn & conc.	"	....
3	Fulton (no lock in Dam No. 4)	11.4	1912	45	"	27.0	12.0	"	"	"	509.0	"	"	....
5	Minetto	5.0	After 1912	45	"	18.0	12.0	"	"	"	500.0	"	"	....
6	High Dam	1.7	do.	45	"	20.0	12.0	"	"	"	500.0	"	"	....
7	Oswego	1.0	1912	45	"	14.5	12.0	"	"	"	517.0	Cut-stn.	"	....
8*	Oswego	0.5	1910	45	"	11.1 max	12.0	"	"	No dam	......	....	....	....

This river was originally canalized by the State of New York in 1820–1828, and the works were rebuilt in 1906 and after for the 12-ft. Barge Canal. New and higher crests were placed on Dams 2, 3, and 7; the other dams are new. Total length from lock No. 1 to lock No. 8, about 20½ miles; total fall, about 119 feet.

* Siphon lock; see p. 448.

OUACHITA RIVER, LOUISIANA AND ARKANSAS.

2	Harrisonburg, La. ..	72.6	Incom.	55	295	.....	6.5	Conc.	Clay	Movable	304	Conc.	Clay	$443,000
4	Monroe, La.	179.0	1911	55	295	.....	6.5	"	"	"	274	"	"	141,000
6	Roland Raft, Ark. ..	238.5	Incom.	55	295	.....	6.5	"	Sand	"	254	"	Sand	193,000
8	Franklin Shoals, Ark.	297.2	"	55	295	.....	6.5	"	Clay	"	234	"	Clay	193,000

Each dam has a pass 135 feet long and a weir, both closed by needles. The weirs vary in length from 50 feet to 110 feet. Dams 2, 4, and 8 have drift chutes from 29 to 39 feet in length, closed by Chittenden drum wickets, with 6.8 feet on the sills. The operation is performed from boats.

NOTE: For locations of rivers see Pl. 46.

ST. MARY'S FALLS CANAL, MICHIGAN.

(See p. 340 and p. 395.)

No.	Location.	Miles from Mouth.	When Completed.	Lock: Available Dimensions. Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
...	Weitzel, Ste. St. Marie	.....	1881	80	485	19.6	17	Stone	Rock	Dam formed by Sault Ste. Marie Rapids				Cost of American locks and canal, St. Mary's Falls, 1853 to 1910, about $8,000,000; of Canadian lock and canal, abt. $5,000,000
...	Poe "	.....	1896	100	745	19.6	22	"	"					
...	Projected "	.....	Incom.	80	1315	19.6	24½	Conc.	"					
...	Canadian Lock, Ste. St. Marie	.....	Abt. 1896	60	900	19.6	22	Stone	"					

SANDY RIVER, MINNESOTA.

No.	Location.	Miles from Mouth.	When Completed.	Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
	Sandy Lake, Libby, Minn...........	1.1	1910	30	158	2.4	9.6	Conc.	Earth	Movable	67.5	Gates with conc. mas'nry	Earth	

TENNESSEE RIVER, TENNESSEE AND ALABAMA.

No.	Location.	Miles from Mouth.	When Completed.	Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
	Hale's Bar,* Tenn..	431.1	Incom.	60	298.0	41.0	6.5	Conc.	Rock	Fixed	1200.0	Conc.	Rock	Cost of Tennessee River locks and dams, $2,063,000 not including works at Hale's Bar
A	Hillsboro, Ala......	286.3	1887	57	275.0	4.5	5.0	Stone	"	"	1290.0	Rock	"	
B	Hillsboro, Ala......	285.0	1887	57	275.0	13.1	2.2	"	"	Rapids Lateral canal past the Muscle Shoals				
1	Florence, Ala......	277.0	1887	57	275.0	10.6	3.1	"	"					
2	Florence, Ala......	275.1	1887	57	275.0	4.0	5.0	"	"					
3	Florence, Ala......	273.2	1887	57	275.0	12.0	5.0	"	"					
4	Florence, Ala......	271.6	1887	57	275.0	10.1	5.0	"	"					
5	Florence, Ala......	269.7	1888	57	283.5	12.1	5.0	"	"					
6	Florence, Ala......	265.0	1889	57	283.5	13.1	5.0	"	"					
7	Florence, Ala......	264.2	1889	57	283.5	11.8	5.0	"	"					
8	Florence, Ala......	263.5	1889	57	283.5	9.9	5.0	"	"					
9	Florence, Ala.	263.2	1889	57	283.5	11.8	3.6	"	"					
	Riverton, Ala......	226.5	1911	80	330.0	26.0	7.0	"	"					

* Under construction by private corporation, 1912.

TRINITY RIVER, TEXAS.

No.	Location.	Miles from Mouth.	When Completed.	Width, Feet.	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
1	Just below Dallas...	499	1910	50	140	11.5	6.0	Conc.	Clay & piles	Movable	99	.....	Clay & piles	
2	....	489	1912	50	140	6.7	6.0	"	"	"	127	.....	"	
4	....	482	1912	50	140	6.7	6.0	"	"	"	127	.....	"	
6	....	470	1911	50	140	14.0	6.0	"	"	"	139	.....	"	
7	....	463	Incom.	50	140	12.2	6.0	"	Rock	"	....	.....	Rock	
...	Hurricane Shoals...	270	"	50	140	18.0	6.0	"	"	Fixed	195	.....	"	
...	White Rock Shoals .	179	"	50	140	11.0	6.0	"	"	Mov.	193	.....	"	

Dam No. 1 is composed of a Chanoine wicket pass 84 feet long and a needle drift chute 15 feet long. (See p. 123.) Dams Nos. 2 and 4 have wicket passes 112 feet long and needle drift chutes 15 feet long, and Dam No. 6 has a wicket pass 124 feet long and a needle drift chute 15 feet long. The White Rock Dam will have a pass and a weir of Chanoine wickets, 116 and 60 feet long respectively, and a needle drift chute 17 feet long. Other locks and dams are proposed.

NOTE: For locations of rivers see Pl. 46.

WABASH RIVER, INDIANA AND ILLINOIS.

No.	Location.	Miles from Mouth.	When Completed.	Lock: Available Dimensions. Width, Feet	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Dam. Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
	Mt. Carmel, Ill.....	92.9	1894	52	214	11.5	3.5	Stone	Rock	Fixed	1100.0	Timber	Rock, gravel	....

WHITE RIVER, ARKANSAS (UPPER).

No.	Location.	Miles from Mouth.	When Completed.	Width, Feet	Length, Feet.	Lift, Feet.	Least Depth on Sills, Ft.	Built of	Built on	Type.	Total Length, Feet.	Built of	Built on	Cost of Works.
1	Batesville.........	300.6	......	36	147	14	....	Conc.	......	Fixed	660.5	Timber	....	....
2	Earnharts.........	309.0	......	36	147	14	....	"	......	"	658.0	"	....	....
3	Walls Ferry.......	321.0	......	36	147	15	....	"	......	"	750.7	"	....	....

NOTE: For locations of rivers see Pl. 46.

WATERWAYS IN EUROPE AND ELSEWHERE.

(For additional information as to sizes of locks, navigable depths, etc., see the Proceedings of the Twelfth International Congress of Navigation, 1912.)

AUSTRIA-HUNGARY.

The principal waterways of Austria-Hungary are the Danube and its affluents, upon which a considerable "open river" traffic is carried on. The most important canalization is that of the Elbe-Moldau system, which affords slackwater navigation on the Moldau below Prague, with a minimum channel depth of 7 feet. Records of traffic on the Elbe and Moldau in the year A.D. 950 are in existence, and these rivers have always been arteries of commerce between Germany and Austria. Locks and dams are also completed or under construction on the Elbe below the mouth of the Moldau, with a view to connecting at Aussig with the open-river navigation of the Elbe. The distance from Prague to Aussig is about 190 miles, and the system when completed will comprise 13 locks (7 on the Elbe and 6 on the Moldau) and 12 movable dams. The lifts of the locks range from 6.2 feet to about 30 feet, the latter being that of the Horin lock, at the lower end of a lateral canal. The earlier locks consisted of one large and one small chamber, placed tandem, the former measuring about $65\frac{1}{2}$ feet in width by 460 feet in length, and the latter about 36 by 220 feet. In most of the later locks the chambers were placed side by side. The dams are all of needles or of gates, hook-needles having been used entirely for the more recent ones, with trestles spaced about 9 feet apart. Raft-chutes, generally 39 feet in width and closed by single sluice gates, are installed at each lock. Construction was begun in 1897.

Two of the dams—those of Libschitz and Mirowitz—have been described and illustrated on p. 610 and after and p. 628 and after.

Canalization, partly with the object of developing water power, is also contemplated for the Elbe above its junction with the Moldau, using some form of sluice gate for the dams.

In addition to the river navigation Austria-Hungary possesses several canals of old construction. A project has been under discussion for many years for a

canal to connect the Danube with the German waterways, but the cost has hitherto been considered prohibitive.

BELGIUM.—See "Holland and Belgium."

CANADA.

The chief waterway of Canada is the St. Lawrence River and its system of lateral canals which connect Lake Ontario with tidewater at Montreal. Lakes Ontario and Erie are connected by the Welland Canal. The general size of the locks is about 260 feet in available length and 45 feet in width, and the maximum draft of boats is 14 feet. The reconstruction of the system is contemplated so as to permit the passage of the largest lake vessels, and an alternative route has been surveyed passing up the Ottawa River and ending in Georgian Bay.

There is also a considerable number of smaller canals with depths from 4 to 6 feet, and there is some open-river navigation on the streams of the north-west.*

ENGLAND.†

The Thames, Weaver, Severn, and minor rivers in England have systems of locks and dams, and there is an extensive system of canals, most of which, however, will accommodate boats of small tonnage only.

The most important artificial waterway is the Manchester Ship Canal, completed about 1894, and connecting the city of Manchester with the tidal channel of the River Dee. The locks are arranged in sets, the chamber widths being 30, 50, and 80 feet, and the lengths 150, 350, and 600 feet respectively. There are four sets in all, besides the tide or guard locks. The average lift per set is $15\frac{1}{8}$ feet, and the total lift to Manchester is $60\frac{1}{2}$ feet. The canal has now a 28-foot depth of water, having been originally built with 26 feet, and is 35 miles in length.

The principal features of some of the Thames locks and dams are as follows:

The river has a system of locks and fixed dams with partly-adjustable weirs. The lock at Teddington, rebuilt about 1900, is between 20 and 25 feet in width and about 600 feet in chamber length, and of concrete. Beside it is an old lock formerly used for skiffs, and with a chamber width of about 7 feet. The dam consists of four fixed weirs, two of which are provided with movable portions closed by Boulé gates resting against fixed iron uprights. The rows of gates have a total height of about 6 feet and the gates are about 7 feet 8 inches long.

The tide lock at Richmond, completed about 1894, is of brick, and has a movable dam closed by Stoney gates and operated from an arch bridge. The gates

* For information about the Canadian canals see the Supplement to the Annual Report of the State Engineer and Surveyor of the State of New York, 1905; Vol. II, p. 1441 and after.

† For information about waterways in Great Britain see the Proceedings, Inst. C. E., Vol. CLXIII, Part I, 1906.

are three in number, each about 66 feet in length and 12 feet in height. Each gate occupies one span of the bridge, and when clear of the water turns on its axis through the action of a cranked arm moving in a slot and lies flat beneath the arch. A fourth span has a fixed weir. The gates are raised daily just before high tide, and lowered an hour or so later.

EGYPT.

The principal navigation in Egypt is on the Nile and connecting irrigation canals. The Nile is partly canalized by movable dams of the bridge type, built chiefly for purposes of irrigation. The system is as follows:

LOCKS.

Location	Assuan.*	Esneh or Isna.	Assiout.†	Zifta.	Rosetta.‡	Damietta.‡
Underlying material	Granite bed-rock	Sand	Fine sand and silt	Fine sand, silt, and clay	Fine sand and silt	Fine sand and silt
Completed	1902	1909	1902	1903	1901	1901
Length between hollow quoins	262′ 6″	262′ 6″	262′ 6″	213′ 3″	213′ 3″	213′ 3″
Clear width	31′ 2″	52′ 6″	52′ 6″	39′ 4½″	55′ 9″	55′ 9″
Lift	65′ 7″ divided among four locks (*)	....	8′ 2″ ord. 12′ 3″ max.	.... About 13 ft.	10′ 9″	10′ 9″
Least depth on miter-sills	9′ 10″	....	6′ 10″	About 6½ ft.	....	....
Walls, height above floor	62′ 6″ max.	....	42′ 9″	32′ 10″	....	....
Material of walls	Rubble faced with cut stn.	....	....	Brick in lime mortar	Riprap solidifi in cement gr	ed by pouring out‡
How filled and emptied	Valves in gates	....	Valves in gates	Valves in gates and culverts	....	....
Lock gates:						
Material	Steel	....	Steel	Steel	....	....
Kind	Rolling; hung from bascule girders	....	Miter	Miter	....	....
How operated	Hydraulic cylinders	....	Double chains and hand winches	....	....	....

* For description see p. 305.
† For description see p. 631.
‡ For description see p. 532 and Proceedings, Institution of Civil Engineers, 1902–1903.

NOTE.—The standard minimum chamber dimensions adopted for locks on the Nile are as follows:

For the delta portion,	39′ 4½″	in width and	213′ 3″	between hollow quoins.
Cairo to Assuan,	52′ 6″	"	262′ 6″	" "
Main irrigation canals,	26′ 3″	"	180′ 6″	" "
Secondary canals,	26′ 3″	"	115′ 0″	" "

DAMS.

Location	Assuan.	Esneh or Isna.	Assiout.	Zifta.	Rosetta.	Damietta.
Type	Pierced Reservoir (*a*)	Bridge, Masonry Arches	Bridge, Masonry Arches	Bridge, Masonry Arches	Fixed	Fixed
Underlying material	Granite bed-rock	Sand	Fine sand and silt	Fine sand, silt, and clay	Fine sand and silt	Fine sand and silt
Material of dam	Rubble in cement mortar	Rubble and concrete	Rubble and concrete See p. 631 and Pl. 73	Brick in lime mortar, rubble, and concrete	Loose rubble with long slopes. Each dam has 2 cut-off walls of grouted riprap about 9½ ft. thick. See pp. 519 and 532	
Length, lock to abutment or abutment to abutment	6400 ft.	Abt. 2830 ft.	2691 ft.	1168 ft.	1640 ft.	1370 ft.
Net length of opening	(*b*)	....	1821 ft.	820 ft.		
Base width	(*a*)	....	Masonry only about 99 ft. Including riprap protection, about 218 ft.	Masonry only, about 99 ft. Including riprap protection, about 225 ft.	200 ft.	200 ft.
Height or thickness	....	masonry floor and foundation 9′ 10″ thick	Same as Esneh	Same as Esneh	24′ 7″ max.	24′ 7″ max.
Closure	(*b*)	(*c*)	(*d*)	(*e*)	....	....
Depth, sill to upper pool	....	....	15′ 1″	19′ 8½″	....	....
Worked from	Stationary hand winches	Traveling winch on roadway	Traveling winch on roadway	Traveling winch on roadway	....	....

(*a*) Dimensions, etc., refer to the dam as originally built; it has since been raised to hold an additional head of 23 feet, increasing its storage capacity nearly 2½ times. This work was begun in 1907. The original pool length was about 140 miles. For description see p. 305.

(*b*) The sluiceways through the dam are closed by 130 Stoney gates of which 100 are 6½ feet wide and 23 feet high and 30 are 6½ feet wide and 11½ feet high; and by 50 sliding gates 6½ feet wide and 23 feet high.

(*c*) The dam has 120 openings or arches of 16 feet 5 inches span, with 108 piers about 6½ feet wide and 11 piers about 13 feet wide. The openings are closed by two ranks of overlapping gates, The roadway across the dam is 19 feet 8 inches in width. The foundation is generally similar to that of the Assiout dam, and consists of a masonry and concrete base 9 feet 10 inches thick with a line of cast-iron sheet piles along upstream and downstream faces, and a riprap-on-clay protection for 60 feet above the dam and a riprap protection for 125 feet below.

(*d*) The dam has 111 openings of 16 feet 5 inches each, with 98 piers 6.6 feet wide and 12 piers 13.2 feet wide. The openings are closed by two ranks of overlapping gates, each 17 feet 5 inches wide and 8 feet 3 inches high. Each gate moves on 4 rollers attached to its downstream side. For description see p. 631.

(*e*) The dam has 50 openings of 16 feet 5 inches each, with 45 piers 6.6 feet wide and 4 piers 13.2 feet wide. The gates are of general design as described in Note (*d*), but are 9 feet 10 inches high each.

FRANCE.*

The waterway system of France is very extensive, and connects practically all parts of the country. It consists of "open" rivers such as the Rhone and the Loire, and of many canalized rivers and canals. The greatest portion of the traffic is in the northeastern district, where the proximity of the coal-fields, has led to the establishment of extensive manufacturing. The canals of this district connect with

* For general information see the Transactions, Am. Soc. C. E., Vol. LIV, Part F, Paper No. 86 (1905).

the waterways of Holland and Belgium, and also with those of Germany. The following are details of some of the chief canalizations.

RIVER SEINE (below Paris).

Location	St. Aubin and Martot, near Rouen (Tide-water limit).	Poses (Amfreville).	LaGarenne (Port-Mort).	Villez.	Méricourt (Sandrancourt).	Meulan (Mézy).	Andrésy-Dénouval.	Bougival (Marly).	Suresnes (just below Paris).
Approximate pool length, miles	9.3	25.4	9.9	14.9	16.2	11.8	17.4	19.2	15.6
Underlying material	....	Chalk	....	Chalk	....	....	....	Sand and gravel	Clay
Lift	7′ 2″ to 10′ 0″	13′ 9″	8′ 7″	7′ 7″	8′ 3″	5′ 9″	9′ 4″	10′ 6″	10′ 9″
Type of dam (all are movable)	Trestle (*a*)	Bridge (*b*)	Bridge	Trestle	Bridge and trestle (*c*)	Bridge and trestle	Trestle	Trestle	Trestle (*d*)
Closed by	Needles on passes, Ch. wickets on weir	Caméré curtains	Curtains	Curtains	Curtains	Curtains and needles	Needles and Ch. wickets	Needles & Boulé gates	Curtains & Boulé gates
Net length of opening	943′ 0″ (*e*)	695′ 6″ (*f*)	737′ 0″ (*g*)	661′ 0″ (*h*)	692′ 6″ (*j*)	1221′ 0″ (*k*)	795′ 6″ (*l*)	444′ 3″ (*m*) movable and 1600′ fixed	664′ 0″ (*n*)
Sill of deepest pass to upper pool	9′ 9″	16′ 5″	13′ 1″	15′ 6″	15′ 3″	13′ 3″	10′ 9″	9′ 9″	15′ 0″

(*a*) In 1906 a project was under consideration for replacing this dam with one of the bridge-and-gate type, of design similar to that used on the Moldau at Mirowitz (see p. 628). The Martot dam was built in 1863–1866 to give a channel depth of 6½ feet, and was raised about 1885 for the present channel depth of 10½ feet.

(*b*) For description see p. 626.

(*c*) Bridges are used for the passes and trestles for the weirs.

(*d*) For description see p. 618.

(*e*) Composed of three navigable passes with a combined length of 510 feet, and of one pass 202 feet long, and one weir 231 feet long.

(*f*) Composed of 5 passes with 16½ feet on the sills, of which the two on the left bank (the locks are on the right bank) are used for navigation. The lengths are: 1 of 91½ feet, 3 of 99½ feet, and 1 of 106½ feet. There are also two weirs each of 99½ feet long with 9 feet 8 inches on the sills.

(*g*) Composed of six navigable passes with a combined length of 586 feet and of one high pass 151 feet long. The six navigable passes have 13.1 feet on the sills.

(*h*) Composed of two navigable passes with lengths of 194½ feet and 198 feet, and of one weir of 268½ feet. The passes have 15½ feet on the sills and the weir 6½ feet.

(*j*) Composed of two navigable passes with a combined length of 205½ feet and a depth of 15.2 feet on the sills; and of two high passes each 243½ feet long with 10.1 feet on the sills.

(*k*). The dam crosses two arms of the river, each with four openings, varying from 114 to 225 feet in net length, and with 10.3 to 13.2 feet on the sills. Total net openings of the arms, 605 and 616 feet.

(*l*) The dam crosses two arms of the river. One arm has three openings each 129½ feet long (with a maximum depth of 10.8 feet on the sills) together with a Chanoine wicket weir 151 feet long; the other arm has a single opening 256 feet long with 10.2 feet on the sill.

(*m*) Composed of one navigable pass 162 feet long; two high passes with a combined length of 282¼ feet; and two fixed weirs with a combined length of 1600 feet.

(*n*) Composed of one navigable pass 237½ feet long closed by alternate bays of gates and curtains (see p. 618); one high pass 203¼ feet long closed by Caméré curtains; and one weir 203¼ feet long closed by Boulé gates. The dam is divided into two portions by an island.

The above system as it now exists was begun about 1878 and was completed about 1888, and forms one of the largest and most important series of movable dams yet built. The least navigable channel depth is $10\frac{1}{2}$ feet. The distance from St. Aubin to the Port-à-l'Anglais dam at the head of the Suresnes pool is about 140 miles, and the total lift is $83\frac{1}{2}$ feet. (For cost see p. 340.) The largest locks of the old system were about 38 feet wide and 370 feet in available length; the new system has at each location one large and one or more small locks. The former are about 55 feet 9 inches wide in the chamber (the entrance width at the gates is about 39 feet) and 497 feet in available length; the latter are about 29 feet and 176 feet in corresponding measurements. The large lock at Bougival is 776 feet in available length, and the one at Suresnes, 526 feet. The depth on miter-sills is usually from 2 to 3 feet greater than the $10\frac{1}{2}$ feet channel depth. Many of the lock walls are faced with brick, with cut-stone trimmings, as on the coping course, vertical corners, etc. Some of the walls have vertical iron fenders. The filling and emptying is done chiefly by short culverts behind the lock gates; in a few cases there are valves in the gates also. Some of the lock gates are of steel, some of steel framing with wooden sheathing, and some entirely of timber. They are all of the mitering type, operated by spars placed just below the coping. (See Fig. 184, p. 482.) Hand power is used in some cases, in others electricity. The Bougival gates are operated by hydraulic pistons. The dams are operated chiefly by hand power, but in some cases electricity is used.

For miscellaneous information see "Dams on the lower Seine," p. 558.

A project was under consideration in 1911 (not, however, recommended by the government engineers) for modifying the system so as to provide a channel between Paris and the sea with a least depth of $23\frac{1}{2}$ feet and a least width of 115 feet. The lowest fixed-bridge clearance would be 74 feet. Only four locks would be used, each $98\frac{1}{2}$ feet wide and 525 feet in available length, with lifts varying from $10\frac{1}{2}$ to 21 feet.

River Seine, above Paris.

Location	Port-à-l'Anglais.	Ablon.	Evry.	Coudray.	Cintanguette.	Vives Eaux.	Melun.	Cave.	Samois.	Champagne.	Madeleine.	Varennes.	Courbeton (Montereau).
Lift	8′ 9″	6′ 0″	5′ 0″	5′ 9″	4′ 9″	4′ 4″	4′ 5″	6′ 9″	6′ 8″	5′ 4″	5′ 5″	4′ 9″	2′ 0″
Approximate pool length, miles	6.4	6.5	5.1	4.1	4.1	3.6	5.0	4.7	5.5	4.0	3.0	2.2	......

The system above Paris was rebuilt and completed about 1881, and affords a least depth of water of $6\frac{1}{2}$ feet. The locks in general are of $39\frac{1}{2}$ feet chamber width and 590 feet available length, except the large lock at Port-à-l'Anglais just above Paris (completed about 1902) which has a chamber width of $52\frac{1}{2}$ feet and

an available length of 590 feet, with about 13½ feet on the miter-sills. Its dam is composed of trestles, curtains, and Boulé gates. The other dams are each composed of a navigable pass closed by Chanoine wickets and a weir closed by needles. The largest wickets are 4 feet 1 inch wide and 11 feet 6 inches long, and most of the wickets are provided with flutter valves 2 feet 1½ inches wide and 3 feet 4 inches long. The maximum depth from upper pool to pass sills is about 11 feet, and to weir sills 7 feet 10 inches.

Size of Locks in France.—By a law passed in 1879, and still in force in 1912, the minimum sizes of new locks, etc., in France were intended to accommodate boats of about 300 tons, and were to be: Chamber width 5.2 meters (about 17 feet 1 inch); available length, 38.5 meters (about 126 feet); depth on miter-sills and in channels, 2 meters (about 6½ feet); and headroom under bridges 3.7 meters (about 12 feet 2 inches). Many locks exceed these dimensions considerably. Thus on the Marne the chamber widths and available lengths are 25 feet 7 inches and 147 feet 6 inches, respectively; on the Petite Saône, from Gray to Verdun, 26 feet 2 inches, and 129 to 139 feet; on the Grande Saône from Verdun to Lyons, from 39½ to 52½ feet and 369 to 505 feet; and on the Yonne from 27 feet 2 inches to 34 feet 6 inches and 305 to 315 feet. The sizes of the large locks of the Seine are given above.

For new canals and waterways a least channel depth of 2½ meters (8.2 feet) is now generally used. This depth was adopted for the Nord and the Nord-est canals (under construction in 1912) and the locks were made 19.7 by 279 feet in available dimensions, accommodating two 300-ton boats. The locks of the Marseilles-Rhone canal, also under construction in 1912, were made 52½ by 525 feet in available dimensions, and provided for a channel depth of 9.8 feet.

Upper Saône.—Gray to Verdun, 9 locks and dams, giving navigation for 67.2 miles with a least channel depth of 6½ feet. The lock chambers are 26 feet 2 inches wide and 129 to 139 feet in available length. The lifts vary from 4 feet 4 inches to 7 feet 2 inches, and the dams are of the needle type, with total lengths of openings varying from 164 to 319 feet.

Lower Saône.—Verdun to La Mulatière dam at Lyons, 5 locks and dams giving navigation for 103.1 miles with a least channel depth of 6½ feet. The lock chambers are from 39½ to 52½ feet wide and from 369 to 505 feet in available length. The lifts vary from 6 feet 8 inches to 9 feet, and the dams have Chanoine wickets on the passes and needles on the weirs except at La Mulatière dam. (See next paragraph.) The average depth from upper pool to pass sills is 11½ feet, and the average length of the passes is 161 feet. The length of the weirs varies from 278½ to 548 feet.

La Mulatière Dam.—This dam is just below Lyons, at the junction of the Saône and the Rhone. (See p. 595.) It was completed in 1882 and is built on

sand and gravel. The lock chamber is $52\frac{1}{2}$ feet wide and 525 feet in available length, and the least channel depth is $6\frac{1}{2}$ feet. The normal lift is 8 feet 3 inches. The pass is 340 feet long with 13 feet 2 inches on the sill and is closed by Chanoine wickets; the weir is $275\frac{1}{2}$ feet long and is closed with Boulé gates.

Marne.—Dizy to Charenton, 19 locks and dams, giving navigation for 111.2 miles with a least channel depth of 7 feet 3 inches. The lock chambers are 25 feet 7 inches wide and $147\frac{1}{2}$ feet in available length. Eleven of the dams have Desfontaines drum weirs with lengths from $98\frac{1}{2}$ to 207 feet, and with needle passes. The other dams have passes of Chanoine wickets. The average depth on the drum weir sills is about $3\frac{1}{2}$ feet. The first drums were built between 1857 and 1867; the last one was installed at Noisiel between 1881 and 1887. (For description see p. 647.)

Yonne.—Auxerre to Montereau, 26 locks and dams giving navigation for 67.2 miles with a least channel depth from 5 feet 3 inches to 6 feet 6 inches. The lock chambers are from 27 feet to 34 feet 6 inches in width and from 305 to 315 feet in available length. At Montereau connection is made with the upper Seine.

Oise.—Janville to the Seine (entering a short distance below Paris), 7 locks and dams, giving navigation for 64.6 miles with a least channel depth of $6\frac{1}{2}$ feet. The total lift is about 33 feet 3 inches, or about 4 feet 9 inches per lock. The old locks are 26 feet 3 inches wide and 151 feet in available length. The new locks are $39\frac{1}{3}$ feet wide, and 410 feet in available length. This river possesses several bridge dams of recent construction, the first two of which, at Isle-Adam and Creil, were finished in 1901. The Creil dam has been described on p. 632.

There are also canalized rivers of minor importance, and a considerable amount of navigation on rivers improved by regulation,* besides an extensive system of canals. A few of the old canals have been enlarged, and some new ones are under construction. (See preceding paragraph: "Sizes of Locks in France.")

GERMANY.

As in France, there is in Germany an extensive system of navigation on "open" and canalized rivers and on canals. Of the first-named the Rhine is by far the most important.† Some of the most important canalized rivers are the following:

Main.—From the mouth to Schweinfurth are 6 locks and dams, viz., at Kostheim, Flörsheim, Okriftel, Höchst, Frankfort, and Schweinfurth. The lifts at Kostheim and Frankfort are 8 feet 10 inches each, the others are 5 feet 11 inches each except at Schweinfurth, where the lift is about 11 feet 9 inches. The pool

* For a description of the Rhone see pp. 72 and after.

† Information regarding the Rhine will be found on pp. 78 and after.

lengths average from 5 to 6 miles. The dams are in general closed by needles. Four of them have Desfontaines drum weirs, 39 feet 4 inches in length and with 5 feet 7 inches of water on the sills. The Schweinfurth dam is of the rolling type (see p. 666) and consists of one cylinder 13½ feet in diameter and 59 feet long, and a second cylinder 6½ feet in diameter and 108 feet long.

Spree.—The Spree has two dams of special interest. One is at Charlottenburg just below Berlin, and was completed in 1885. Its pass is closed by a very large Desfontaines drum wicket, 32 feet 9 inches in length and with a depth of 9 feet 2 inches on the sill and a lift of about 4 feet (see Fig. 230, p. 647). The weir is closed by Boulé gates moving on fixed uprights supported against an overhead bridge. The other dam is the Mühlendamm, in the city of Berlin, and consists of 3 openings of about 50 feet each center to center of piers, subdivided by fixed iron uprights about 8½ feet centers on which move gates on rollers. (See Fig. 220, p. 615.) The depth on the sill is 12 feet and the lift is about 6 feet. The lock is 37 feet wide and 378 feet between hollow quoins, with 7½ feet on the miter-sills. The present work was built between 1890 and 1893.

Oder.—Above Breslau are 12 locks with lifts from 4 feet 3 inches to 13 feet 6 inches, extending over about 43 miles. The chambers are 31½ feet by 590 feet, and the least channel depth is 5 feet.

Netze (Oder-Weichsel system).—This river has 7 locks extending over about 48 miles. The chambers are 31½ by 180 feet, and the least channel depth is from 5 to 6½ feet.

Ems.—This river was canalized in connection with the Dortmund-Ems Canal between Meppen and the mouth, and has 5 locks with movable dams. The works were completed in 1899. The locks on canal sections have entrance widths of 28¼ feet, available lengths of 220 feet, and depths on sills of 10 feet. On the Ems the entrance widths are 33 feet, available lengths 541⅓ feet, and depths on sills, 10 feet. The lifts on canal sections vary in general from 11 to 13½ feet, although 2 locks have lifts of 20⅓ feet each (at Münster and Gleesen); and at Henrichenburg is the mechanical lift-lock covering a difference of level of about 46 feet.* The least channel width in the Ems is 98½ feet.

Of the dams, 4 are of the needle type, with hook-needles 3½ by 3½ inches. The fifth dam (at Herbrum) is of Stoney gates, six in number, each of 28 feet clear span by about 8 feet high, and operated by racks and pinions. The lift is about 6½ feet. The average lift per dam is 6¾ feet, the total lift of the 5 dams being 33⅔ feet.

Locks and dams are also to be found on the Elbe, Weser, and other streams. There is also a considerable amount of traffic on rivers improved by regulation.

* A lock of ordinary type of a single lift has been constructed at the same point in connection with the enlargement of the canal. It has five side-ponds whose combined capacity is twice that of the lock chamber, and which save about 76 per cent of the water.

Among the most important canals (some of which were under construction in 1912) are the Elbe-Trave Canal (see p. 449) with its system of siphon locks; the Teltow Canal near Berlin, having a length of 24 miles, and with one twin siphon lock with lifting gates; the Dortmund-Ems Canal referred to above under the River Ems; the Oder-Spree Canal, with a length of 55 miles and having certain of its locks of the siphon type; the Rhine-Weser Canal; and the Berlin-Stettin Canal, the length of which is 61 miles. In all new construction the locks and prisms for waterways west of Berlin are made to suit 600-ton boats; east of Berlin they are made for 400-ton boats, except on the Berlin-Stettin Canal which will take boats of 600 tons. The drafts of each type of vessel is about 6 feet; the headroom under bridges is 13 feet 1½ inches (4 meters) or more; and the curves have a least radius of about 3000 feet except in special cases. The widths of the lock chambers vary from 28.2 to 62.5 feet, and the lengths from 191 to 706 feet. The depths of the lower miter-sills vary from 8.2 to 14.7 feet. Reinforced concrete has been used on the new canals very extensively for locks, bridges, aqueducts, etc.

HOLLAND AND BELGIUM.

Holland and Belgium have a large mileage of waterways, comprising the mouths of the Rhine and other rivers, and numerous canals. The principal canalized river in Belgium is the

River Meuse.—The system of locks and dams rebuilt between 1874 and 1880 affords a least channel depth of 7 feet. The dams are movable, and consist of needle passes with average lengths of 142 feet and an average depth on the sills of 10.2 feet, and of Chanoine wicket weirs with average lengths of 180 feet and an average depth on the sills of 7½ feet. The average lift is 8 feet.

It may be noted that on the Lower Saône and the Upper Seine, referred to elsewhere, the opposite arrangement was followed, wickets being used for the passes and needles for the weirs. On the Big Sandy River in the United States the same arrangement was adopted as on the Meuse.

Most of the existing canals accommodate boats of about 300 to 350 tons burden. The largest of the new canals will provide for boats of 1000 tons. Among the most important ones of recent construction are the following:

Amsterdam Ship Canal (Holland).*—The Ymuiden lock, finished in 1894, has a chamber width of 82 feet, a length of 741 feet (divided by intermediate gates into lengths of 231 and 510 feet), and a least depth on sills at low tide of 30½ feet. This lock was designed to take vessels of 78 feet beam, 722 feet length, and 30½ feet draft.

The older tide lock at Ymuiden has a chamber 59 feet wide, 394 feet long

* For information on the Holland ship canals see Transactions, Am. Soc. C. E., Vol. LIV, Part F, No. Paper 85 (1905).

(divided by intermediate gates into lengths of 146 and 230 feet), and with a low-tide depth on sills of $23\frac{1}{3}$ feet.

Ghent-Ter Neuzen Canal (Belgium).—The Ter Neuzen lock, finished in 1905, has a chamber width $80\frac{1}{2}$ feet at coping level, a width between entrance walls of 59 feet, an available length of 460 feet, and a depth on the lowest sill of $27\frac{1}{2}$ feet. The Sas of Ghent lock dimensions are: Chamber width, 128 feet (maximum); between entrance walls, 86 feet; available length, 656 feet; depth on sills, $31\frac{1}{6}$ feet.

Merwede Canal (Holland).—The latest locks are from $95\frac{1}{6}$ to 116 feet in maximum chamber width, $39\frac{1}{2}$ to 46 feet between entrance walls, and 394 feet in available length. They were built for Rhine barges of 8 feet draft. Maximum lift, about $7\frac{1}{4}$ feet.

Brussels Ship Canal (Belgium).—This connects Brussels with Scheldt, and has a length of about $18\frac{1}{2}$ miles, with a depth of 21.2 feet. There are 3 locks with available dimensions of $52\frac{1}{2}$ by 374 feet, and 21.2 feet on the sills. The total lift above mean low tide is about 45 feet.

In addition to the foregoing some of the old canals are being enlarged and some new ones will be built.

ITALY.

The waterways of Italy, owing to the configuration of the country, are not extensive, and traffic is chiefly confined to open-river navigation on the Po, the Arno, and the Tiber, and to a few canals of small dimensions. A modernization and extension of certain of the waterways is to be undertaken, however, chiefly in the basin of the River Po, with the object of connecting Venice with Milan and the Italian lakes. This system will have a total length in river and canal of 302 miles, and will overcome a maximum elevation (between sea-level and Lake Como) of 660 feet. The largest of the new locks will be 33 by $233\frac{1}{2}$ feet in available dimensions, with $8\frac{1}{4}$ feet on the sills, and the canal section will be 59 feet wide on the bottom and designed for 600-ton boats.

PANAMA CANAL.

This canal crosses the Isthmus of Panama between the cities of Panama and Colon, and has a length from shore line to shore line of about $41\frac{1}{2}$ miles and of about 50 miles if the harbor entrances are included. It is designed for the largest modern vessels, and has a least bottom width of 300 feet and a least depth of 40 feet. The total normal lift from tidewater to the summit level is 85 feet. On the Atlantic side this is overcome by a flight of 3 twin locks at Gatun of equal lifts, and on the Pacific side by one twin lock at Pedro Miguel with a lift of $30\frac{1}{3}$ feet and 2 twin locks at Miraflores, making 12 the total number of locks. The

width of the chambers is 110 feet, the maximum available length 1000 feet, and the minimum depth on the sills, $41\frac{2}{3}$ feet. An intermediate set of gates divides the chambers into lengths of about 400 and 600 feet, and there are also double sets of gates at each end. The smallest lock gate is $46\frac{1}{3}$ feet in height and the largest is 82 feet, its weight being about 730 tons. All are of steel and are of the straight mitering type, and operated by spars and bull wheels (see p. 484). The length is about 65 feet and the mid-depth about 7 feet.

Each side of a twin lock contains one circular culvert 18 feet in diameter, running the full length of the lock and having an area of 254 square feet. This connects at right angles with 11 lateral culverts, each with an area of 41 square feet, and these in turn open upwards through the chamber floor in five openings each 12 square feet in area. The side-wall culverts are intended for use only with the 400- and 600-feet chambers. The middle-wall culvert is also 254 square feet in area and runs the full length of the lock, communicating with the adjacent chambers through 10 lateral culverts on each side and each with an area of 33 square feet. These in turn open upwards through the floor, and each one is provided with a cylindrical valve about $6\frac{1}{2}$ feet in diameter, for separate control. The main culverts are controlled by duplicate Stoney valves, with outside dimensions of 10 by 20 feet, acting under a maximum head of 60 feet, equivalent to a pressure of 270 tons. All masonry is of concrete and all operation is electrical.

Guard or fender chains of metal 3 inches in diameter are placed at certain vital points to prevent ships from striking the lock gates. They are hoisted by hydraulic pistons across the entrances when in use, and are lowered to the bottom for the passage of ships. Emergency dams, consisting of swing bridges with uprights and gates (generally similar in type to those of river bridge dams), are also provided for use in case of disaster to the lock gates.

The summit level will be fed from an artificial lake about 164 square miles in area, created by damming the Chagres River. For this purpose an earth-and-rock dam was built at Gatun, made from the prism excavation and with a total crest length of about 7500 feet, a base width of about 2100 feet, a width at normal water level of about 400 feet, and a top width of 100 feet. The top is about 30 feet above normal water level. The dam contains about 20,000,000 cubic yards. Near the middle, in a natural hill, a spillway of concrete masonry with Stoney gates was constructed, its sill being 16 feet below the normal water surface and having a discharge area of 14 openings each 45 feet in width. This spillway is capable of passing about 154,000 cubic feet per second.

The total amount of excavation required to complete the canal was estimated at about 225,000,000 cubic yards, and the total amount of concrete at about 1,500,000 cubic yards. The total cost, including all expenses, was estimated at about $375,000,000.

RUSSIA.

Russia has several extensive systems of natural waterways, the principal of which is the Volga and its tributaries.* The river navigation is chiefly "open," although a few rivers, such as the Moskva and the Sheksna are canalized. There are also several canals uniting most of the river and lake systems. On the Moskva are 7 locks with available dimensions of $55\frac{3}{4}$ by $672\frac{1}{2}$ feet; on the Oka are two of $55\frac{3}{4}$ feet by 873 feet; and on the Sheksna there are locks 42 by 1050 feet. The average maximum draft of boats is stated to be from 6 to $6\frac{1}{2}$ feet. On the larger rivers the draft can be increased in times of flood to 16 feet or even more, and the largest boats with such a draft will carry nearly 2000 tons.

SWEDEN.

In comparison with other countries, the waterway system of Sweden is not of great extent. It is composed of rivers, lakes, and canals, many of which are connected directly with the sea. One of the most important links consists of the Trollhattan and Gotha canals, which form an inland waterway by canal, river, and lake between the Baltic and the Cattegat. The present locks upon it are $24\frac{1}{3}$ feet wide and 117 feet in available length, with a depth of $9\frac{3}{4}$ feet on the sills. It is expected to reconstruct the Trollhattan section for vessels with a draft of 13 feet, but with the locks designed for a draft of $16\frac{1}{4}$ feet.

* A description of the Volga will be found on pp. 84 and after.

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PLATE 45a

PLATE 45b

PLATE 45b

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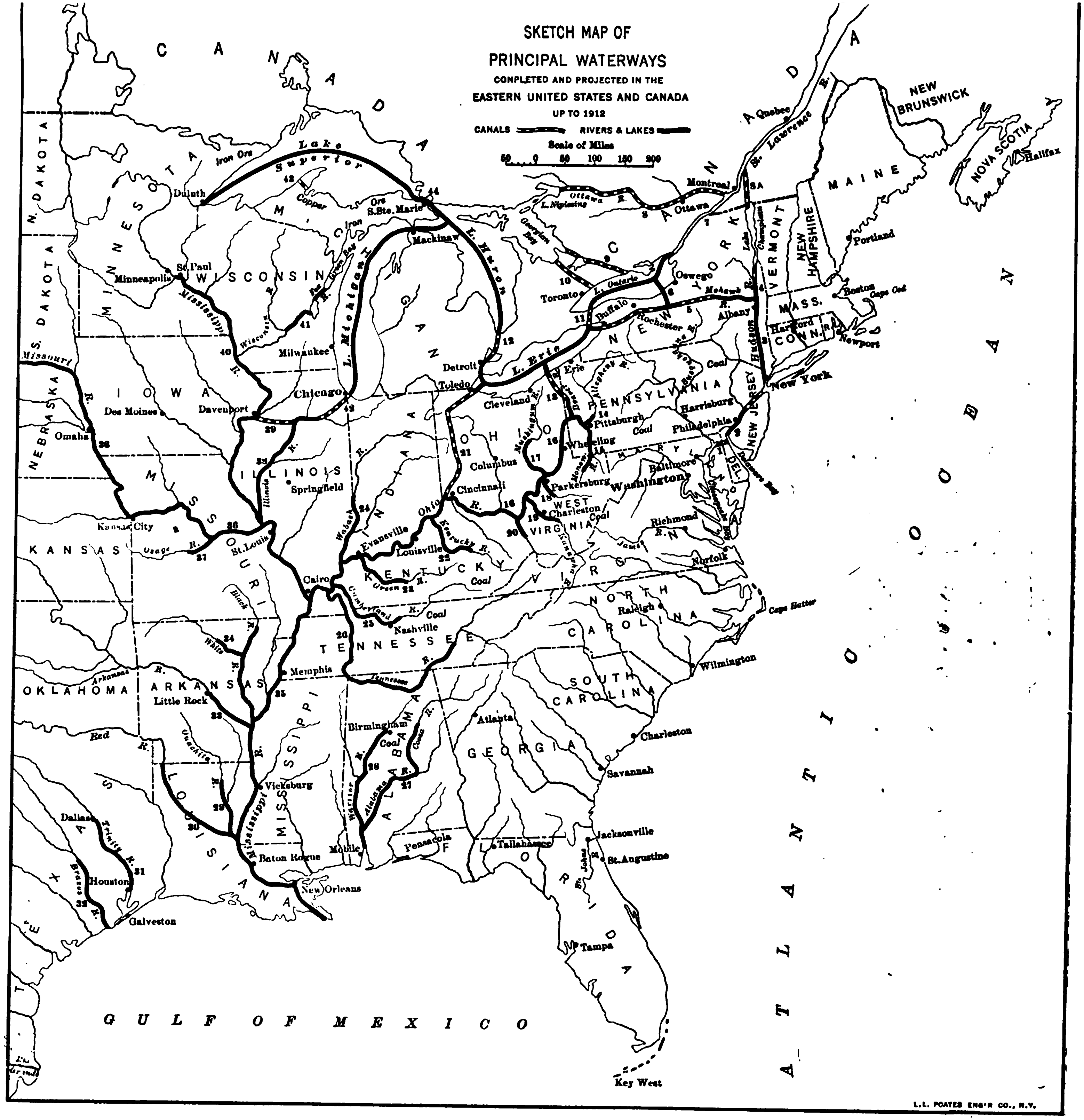
1. Chesapeake & Delaware Ship Canal (project); 14 miles, 2 locks.
2. Delaware & Hudson Ship Canal (project); canal proper 31 miles, 4 locks.
3. Lower Hudson River, New York to Troy; open river navigation, 12 ft. depth.
4. Upper Hudson, Troy to Fort Edward; canalized for New York State Barge Canal; 7 locks, 7 fixed dams; 12 ft. depth. (See page 691.)
5. New York State Barge Canal, for 2500-ton barges; in part a reconstruction of the Erie Canal. Includes the canalization of the Mohawk and other rivers by movable (needle and bridge-and-gate) dams, and by fixed dams. Begun in 1905; 12 ft. depth. (See pp. 695 and 696.)
6. Oswego River; part of New York State Barge Canal system; 12 ft. depth. (See p. 698.)
7. St. Lawrence River and canals; 14 ft. least depth.
8. Georgian Bay & Tidewater Ship Canal; 20 ft. depth. (Project.)

8*a*. Richelieu Canal; connects with Hudson River via Lake Champlain and New York State Barge Canal.

9. Trent Canal, under construction; 6 ft. depth.
10. Lake Ontario & Georgian Bay Ship Canal. (Project for 22 ft. depth.)
11. Welland Canal; 14 ft. depth.
12. St. Clair Flats Ship anal; 21 ft. depth.
13. Lake Erie & Ohio River Barge Canal, Pittsburgh to Ashtabula, Ohio; 130 miles. (Project.)
14. Allegheny River, canalized for 25 miles; 7 ft. depth, 3 dams. (1 movable dam of Chanoine wickets and bear-traps, and 2 fixed dams. See p. 688.)
15. Monongahela River, canalized for 130 miles; 15 fixed dams, 6 to 9 ft. depth. (See p. 694.)
16. Ohio River, canalization in progress; 9 ft. depth, movable dams. (Chanoine wickets and bear-traps; 12 completed by 1912. See p. 697.)
17. Muskingum River, canalized for 84 miles; 10 fixed dams, 5½ ft. depth. (See p. 694.)
18. Little Kanawha River, canalized for about 50 miles; 5 fixed dams, 4 ft. depth. (See p. 692.)
19. Kanawha River, canalized for 93 miles; 6 ft. depth, 10 dams. (8 movable, of Chanoine wickets, and 2 fixed. See p. 691.)
20. Big Sandy River, canalization in progress by movable dams of needles and Chanoine wickets; 5 dams completed, 6 ft. depth. (See p. 688.)
21. Lake Erie & Ohio River Barge Canal, Cincinnati to Toledo, 246 miles. (Project.)
22. Kentucky River, canalized by fixed dams; 6 ft. depth. (See p. 692.)
23. Green and Barren Rivers, canalized for 250 miles; 7 fixed dams, 4 to 6 ft. depth. (See p. 690.)
24. Wabash River. (Project. See p. 700.)
25. Cumberland River, improvement on about 400 miles in progress; canalization and dikes. (See p. 689.)
26. Tennessee River, improvement on about 651 miles in progress; canalization and dikes. (See p. 699.)
27. Coosa and Alabama Rivers, canalization in progress with fixed dams. (See p. 689.)
28. Black Warrior River, canalization in progress with fixed dams. (See p. 688.)
29. Ouachita River, Louisiana, canalization in progress; needle dams. (See p. 698.)
30. Red River, partly improved by regulation.
31. Trinity River, Texas, canalization in progress by movable dams. (Boulé gates, Chanoine wickets, and needles. See p. 699.)
32. Brazos River, Texas, canalization in progress by movable dams. (Chanoine wickets and bear-traps.) 6 ft. depth. (See p. 689.)
33. Arkansas River, partly improved by regulation.
34. White River, canalization in progress by fixed dams. (See p. 700.)
35. Lower Mississippi, Cairo to New Orleans, about one thousand miles; minimum depth 9 ft. Dredging and protection work.
36. Missouri River; partly improved by regulation and bank protection. (See Part I, Chaps. II, IV, and V.)
37. Osage River, partly improved. (See p. 698.)
38. Illinois River, partly canalized; 6 ft. depth. (See p. 691.)
39. Illinois & Mississippi Canal; 7 ft. depth.
40. Upper Mississippi River, partly improved by regulation. (See Part I, Chap. II, and also p. 692.)
41. Fox River, canalized prior to 1860; 4 to 6 ft. depth. (See p. 690.)
42. Chicago Drainage Canal, Chicago to Joliet; 21 ft. depth.
43. Portage Lake Ship Canal, 21 ft. depth.
44. St. Mary's Falls Ship Canal, 21 ft. depth. (See p. 699.)

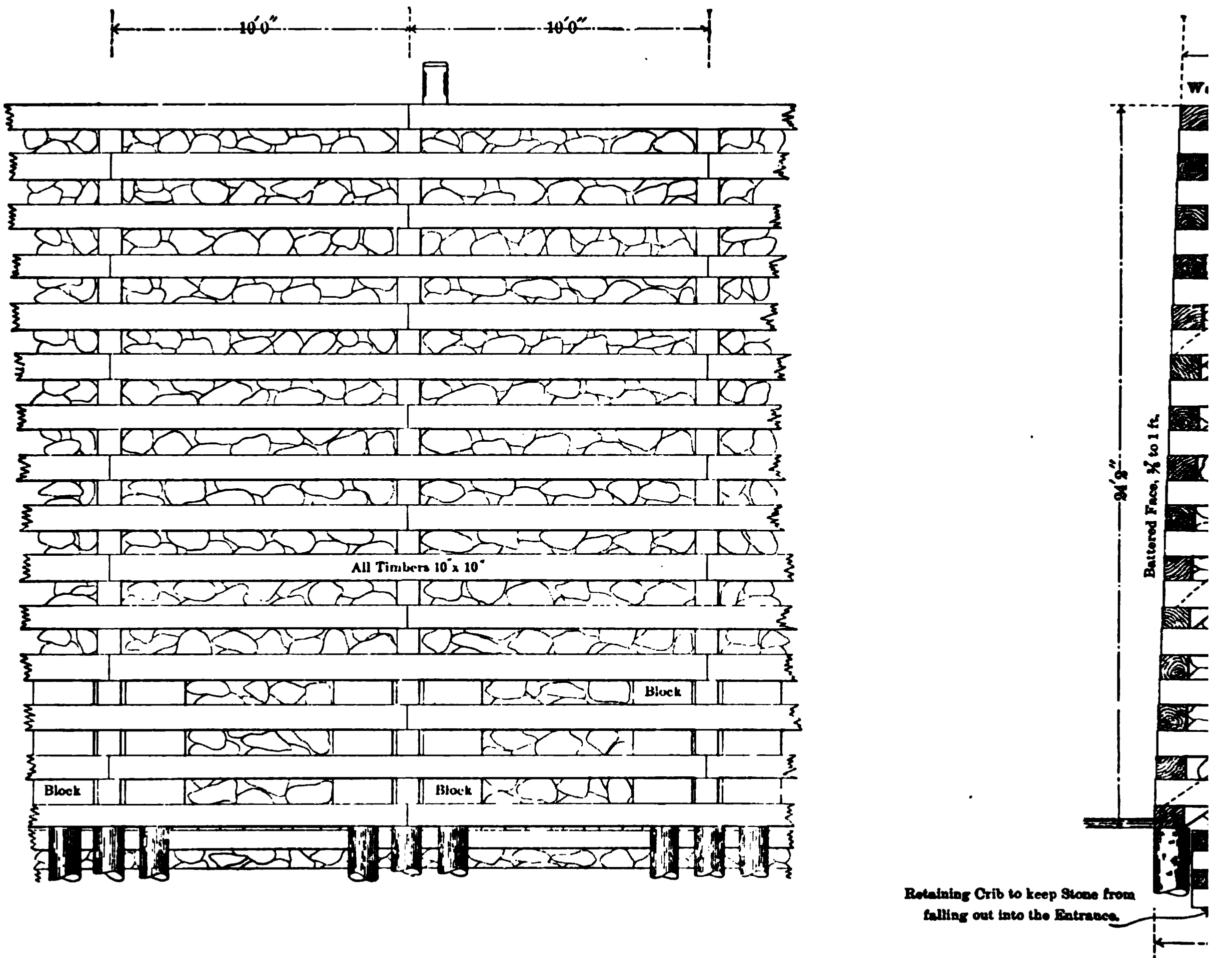
NOTE—In addition to the above many of the smaller rivers have been partly improved by dikes or by dredging, and some have one or more locks and dams.


SKETCH MAP OF
PRINCIPAL WATERWAYS
COMPLETED AND PROJECTED IN THE
EASTERN UNITED STATES AND CANADA
UP TO 1912
CANALS
RIVERS & LAKES
Scale of Miles
50 0 50 100 150 200
CANADA
ATLANTIC OCEAN
GULF OF MEXICO
L. I. POATES ENGR CO., N.Y.

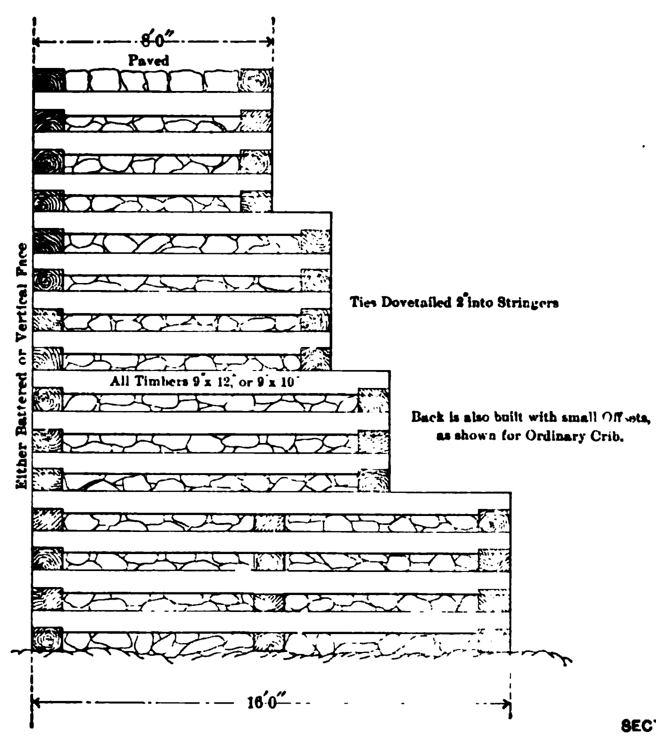
PL. 46.

PLATE 46

PLATE 46a



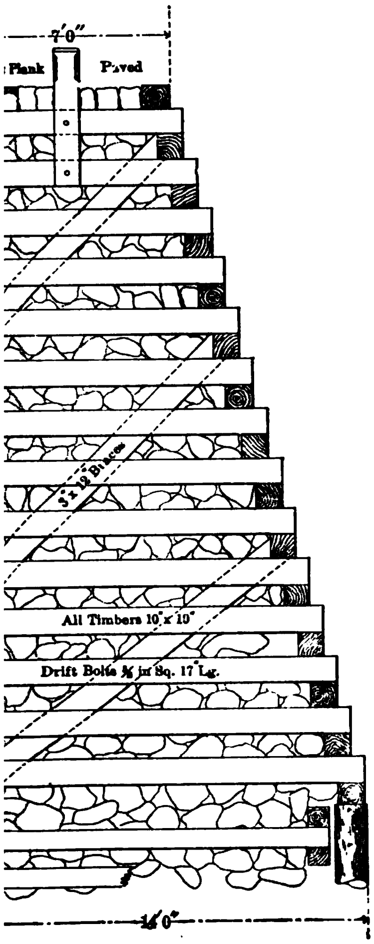
ELEVATION AND SECTION OF ORDINARY CRIB.

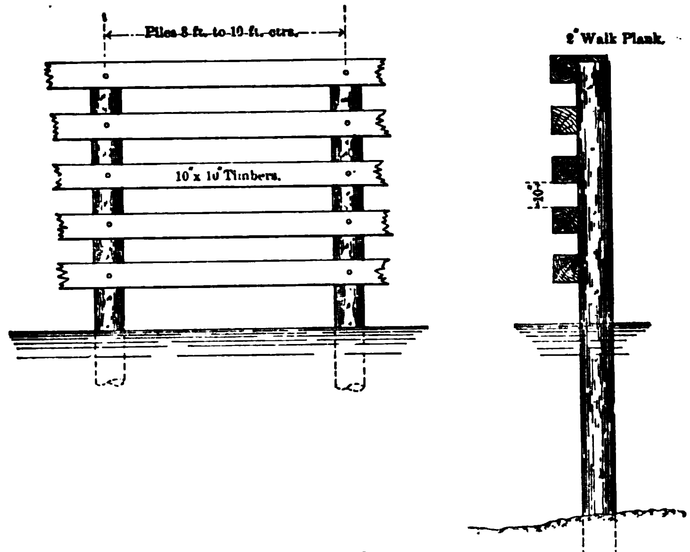


SECTION OF FRAMED CRIB.

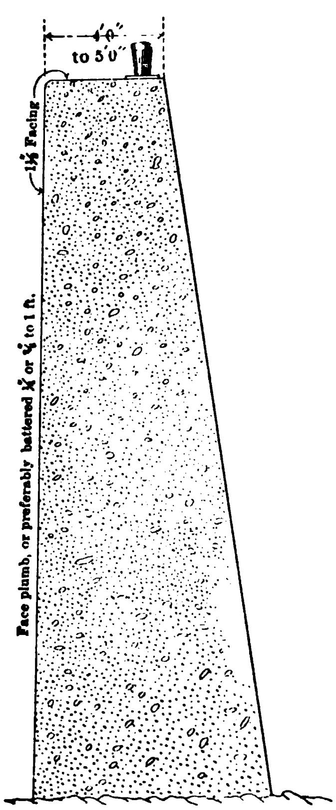
SECTION OF CONCRETE GUARD CRIB (FOR RIVER

For Guide Cribs wider bases are used if there is earth





PILES WITH TIMBER FACING



S·DE)
…hind.)

TYPES OF GUIDE CRIBS AS USED ON
TRIBUTARIES OF THE OHIO RIVER.

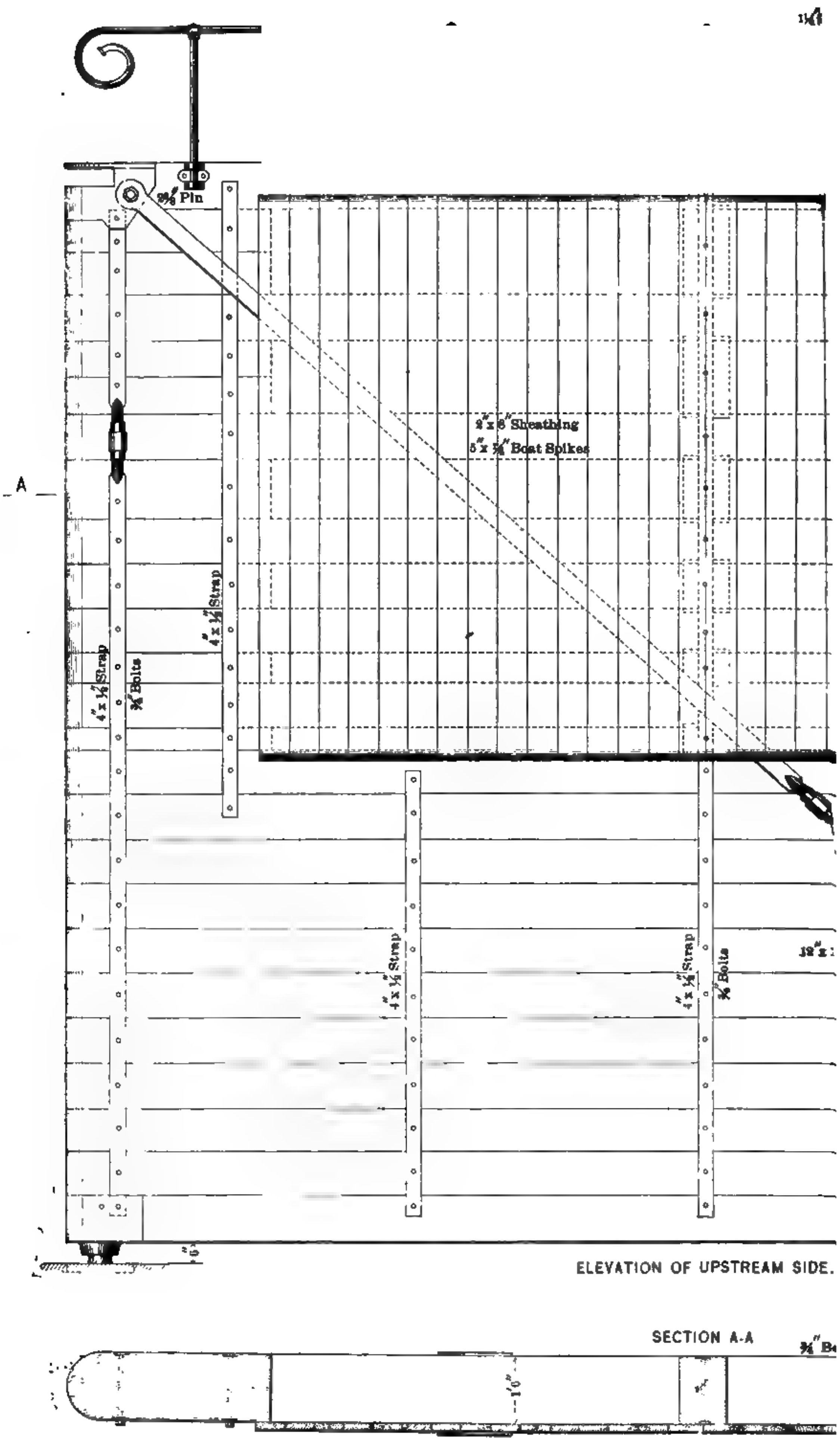
Scale:
12" 0' 1' 2' 3' 4'

NOTE.—Guide cribs are below and above land walls of locks. Guard cribs are below and above river walls of locks. In several recent examples of unsupported walls where the traffic was in large barges the masonry has been bolted to the rock.

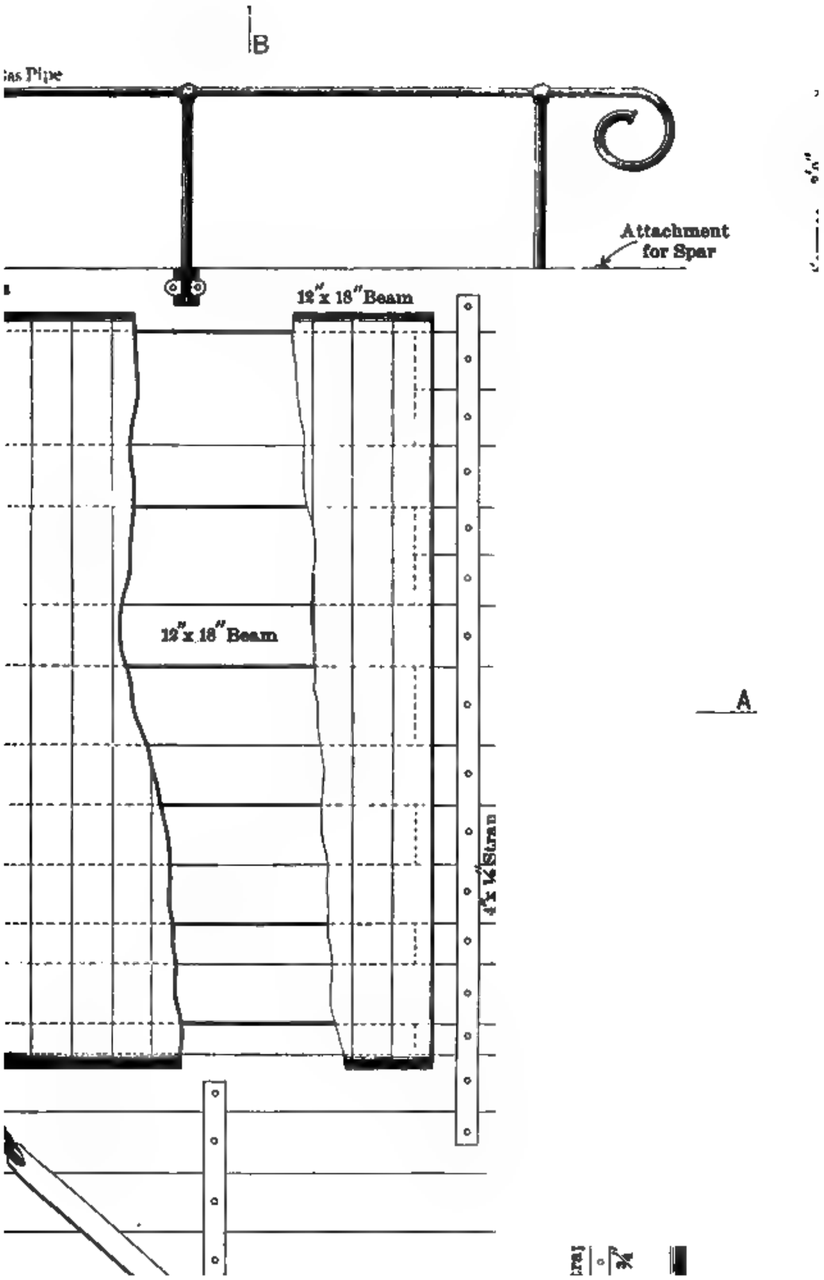
PL. 46a.

(Reference, p. 420 and after.)

PLATE 47



ELEVATION OF UPSTREAM SIDE.



SECTION B-B

DESIGN FOR AN 18-INCH
WOODEN LOCK-GATE.

Scale:
12″ 6″ 0′ 1′ 2′ 3′

Pl. 47.

(Reference, p. 464 and after.)

PLATE 48

PL. 49.

(Reference, p. 469.)

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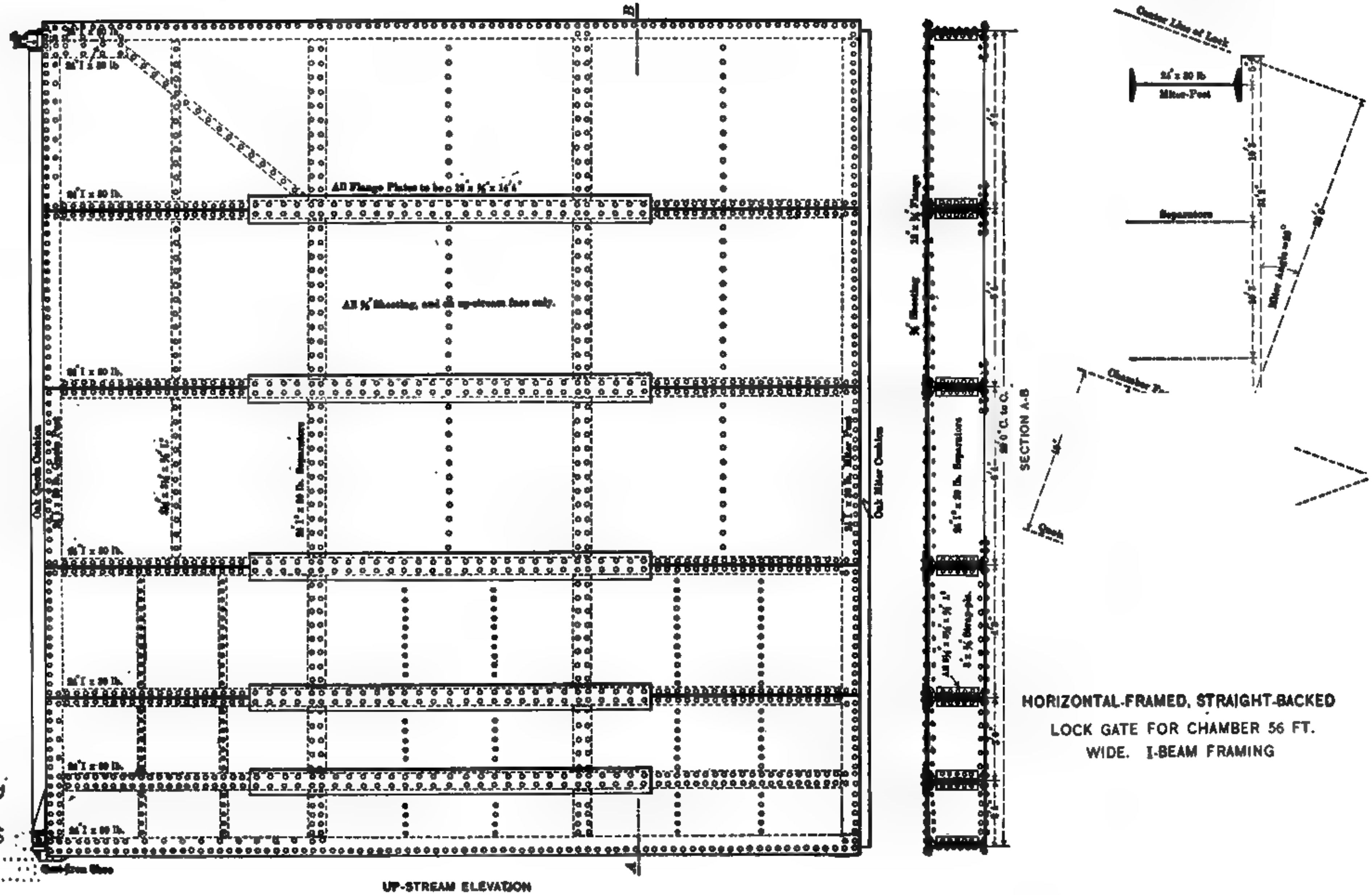
PLATE XVIII. VOL. LIV. PART F.
TRANS. AM. SOC. CIV. ENGRS.
INTER. ENG. CONG., 1904.
SIBERT ON
ARTIFICIAL WATERWAYS IN THE UNITED STATES.

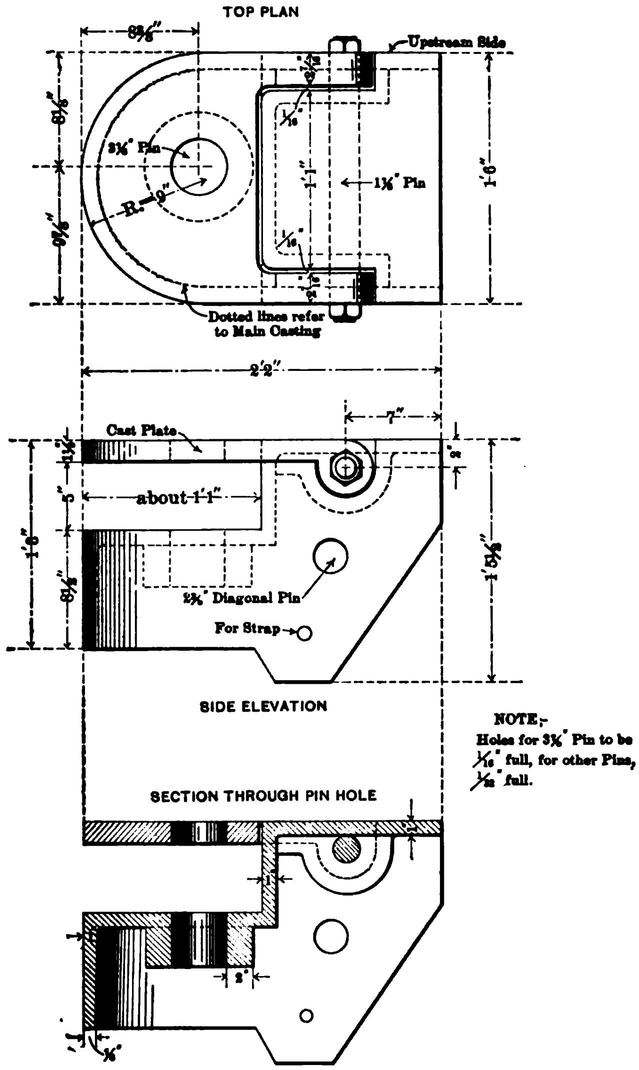


HORIZONTAL-FRAMED, STRAIGHT-BACKED
LOCK GATE FOR CHAMBER 56 FT.
WIDE. I-BEAM FRAMING

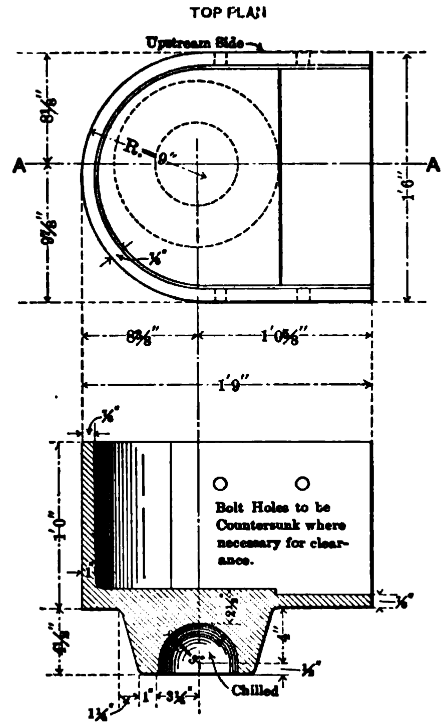
PL. 49.

(Reference, p. 469.)

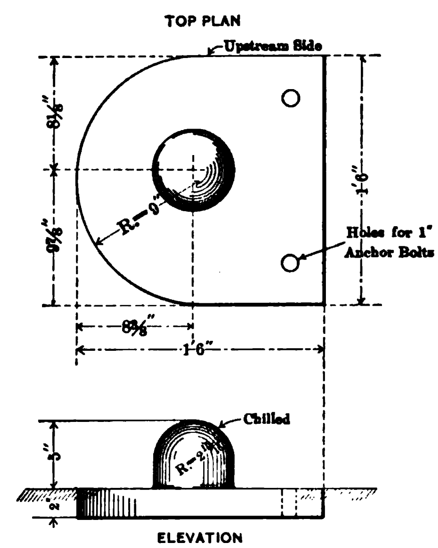
PLATE 50



CAST BONNET WITH HINGED CAP-PLATE FOR AN 18-INCH GATE.

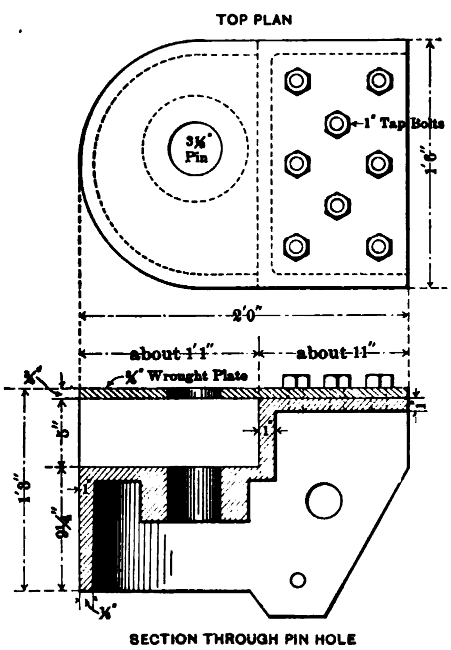


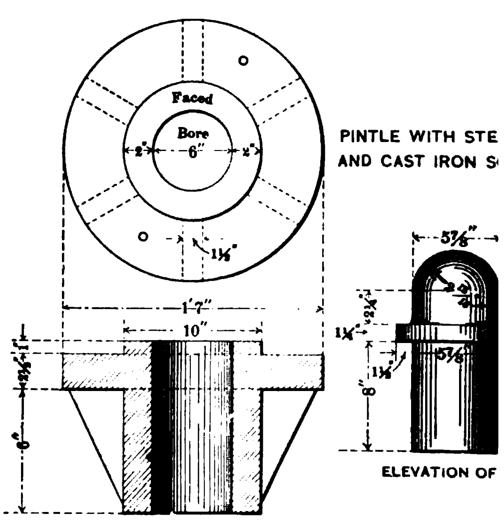
CAST SHOE FOR AN 18-INCH GATE.



PINTLE FOR AN 18-INCH GATE, IN ONE PIECE.
(For a 12-inch sill)

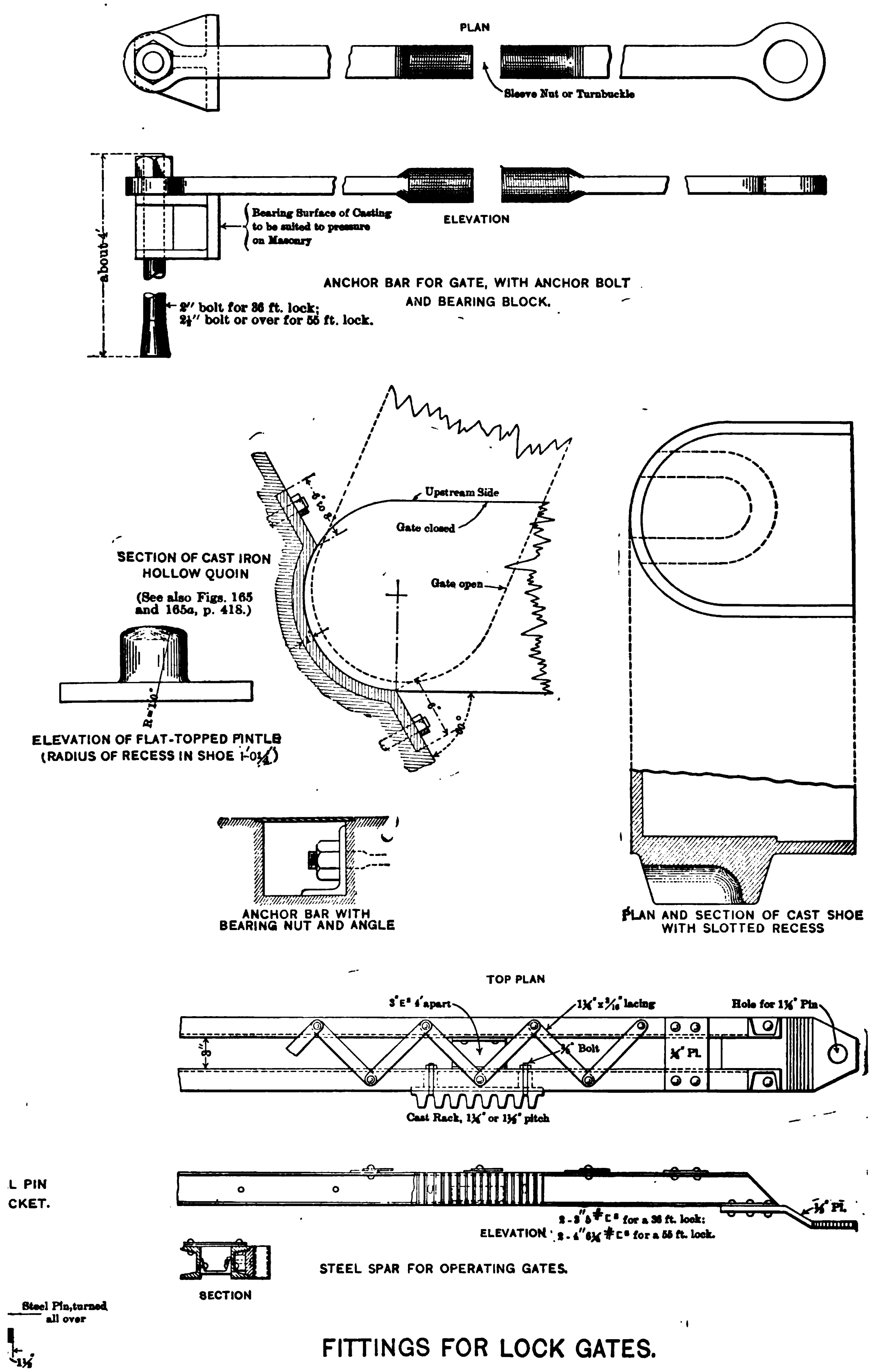
CAST BONNET WITH BOLTED CAP-PLATE.



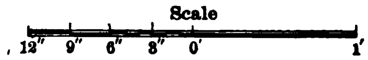


PINTLE WITH STE
AND CAST IRON S

PLAN AND SECTION OF SOCKET



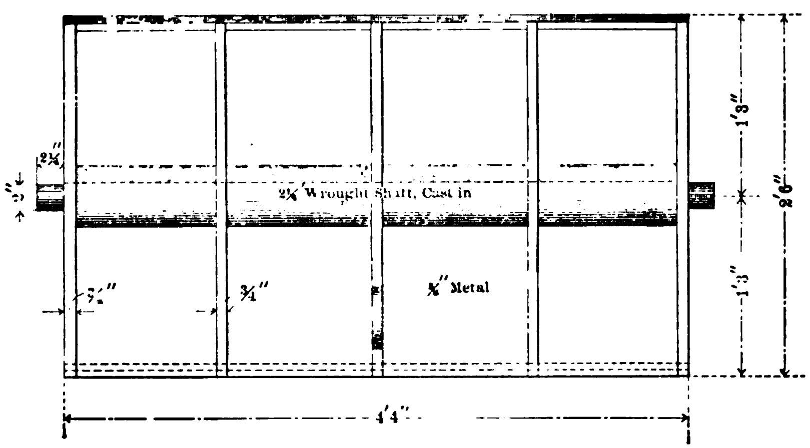
FITTINGS FOR LOCK GATES.



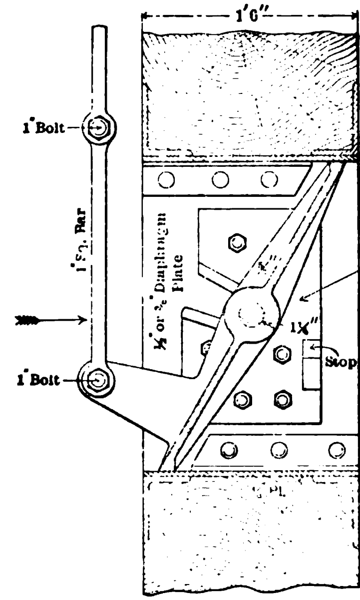
Pl. 50.

(Reference, pp. 476 and 482 and after.)

DETAILS OF BALANCED VALVES, AS USED IN LOCK-GATES.

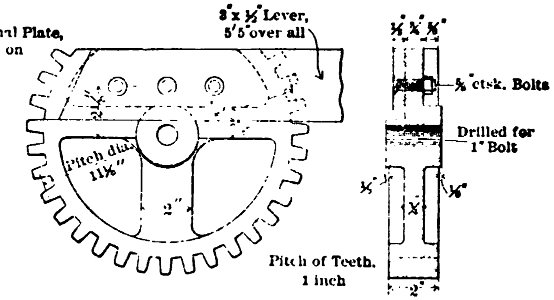


ELEVATION OF CAST IRON VALVE.



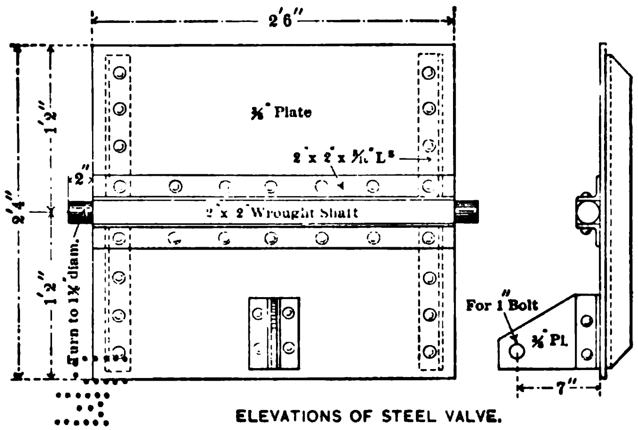
SECTION OF WOODEN GATE, SHOWING VALVE AND GIRDER IN PLACE.

This style of valve is also used in a reversed position, with operating rods arranged to push down for opening, instead of pulling up as shown.

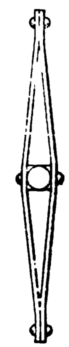


ELEVATION. SECTION.

DETAILS OF SPUR-WHEEL, used in connection with Rack, for operating Gate Valves.

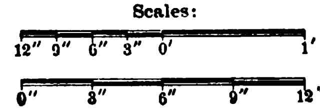


ELEVATIONS OF STEEL VALVE. (of size for high lifts.)



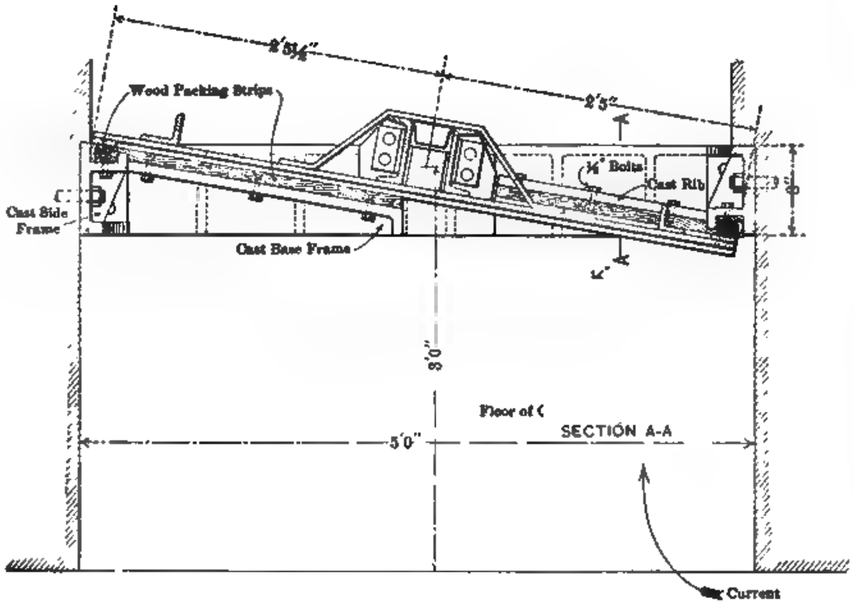
END ELEVATION OF VALVE FORMED OF TWO STEEL PLATES, BENT.

Scales:

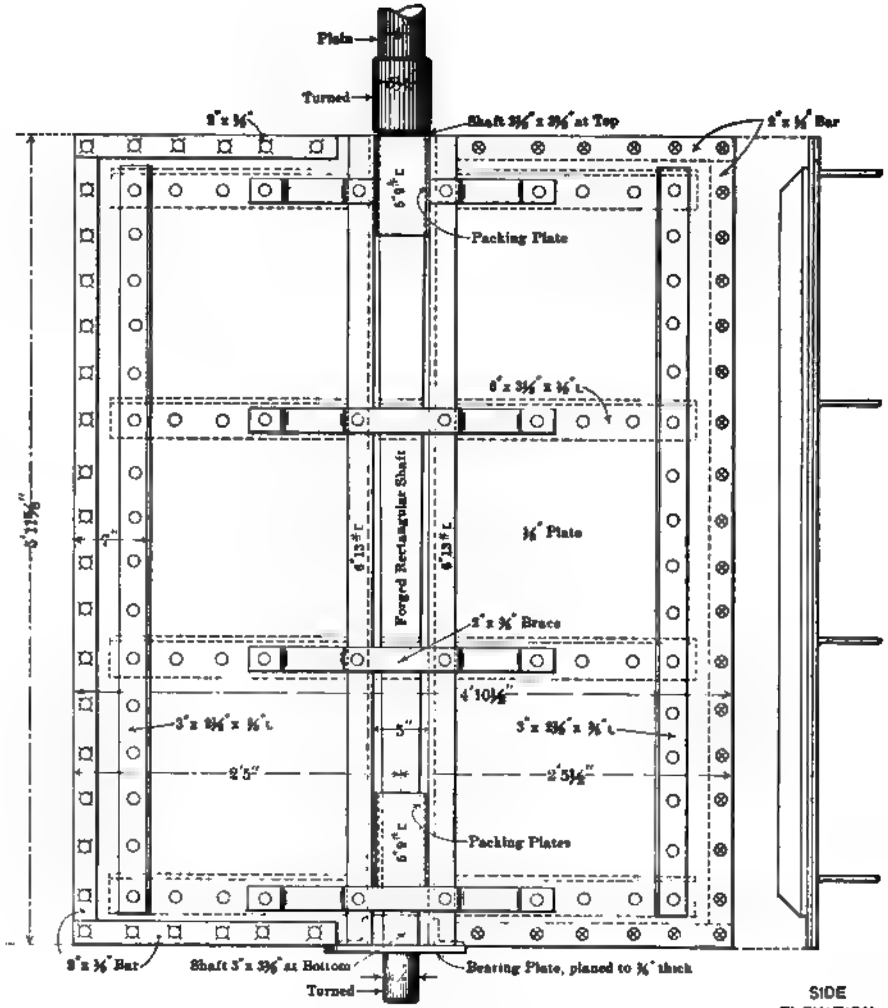


Pl. 51.

(Reference, p. 494.)



SECTION OF CULVERT SHOWING WICKET AND BASE AND SIDE FRAMES IN POSITION.
(6" Angles omitted.)



REAR ELEVATION OF WICKET SHOWING SHAFT IN PLACE.

SIDE ELEVATION.

BALANCED WICKETS OF STEEL, FOR CULVERTS, AS USED ON GREEN AND KENTUCKY RIVERS, KY.
(1897-1900.)

Scale

12" 9" 6" 3" 0 1'

The operating machinery for these wickets consists of a train comprising two gear wheels of 39 inches pitch diameter and two pinions of $7\frac{13}{16}$ inches pitch diameter, all of $3\frac{1}{2}$-inch face and $1\frac{3}{4}$-inch pitch of teeth. For heads up to about 14 feet one man can operate a wicket without difficulty.

PL. 52.

(Reference, p. 495.)

MU

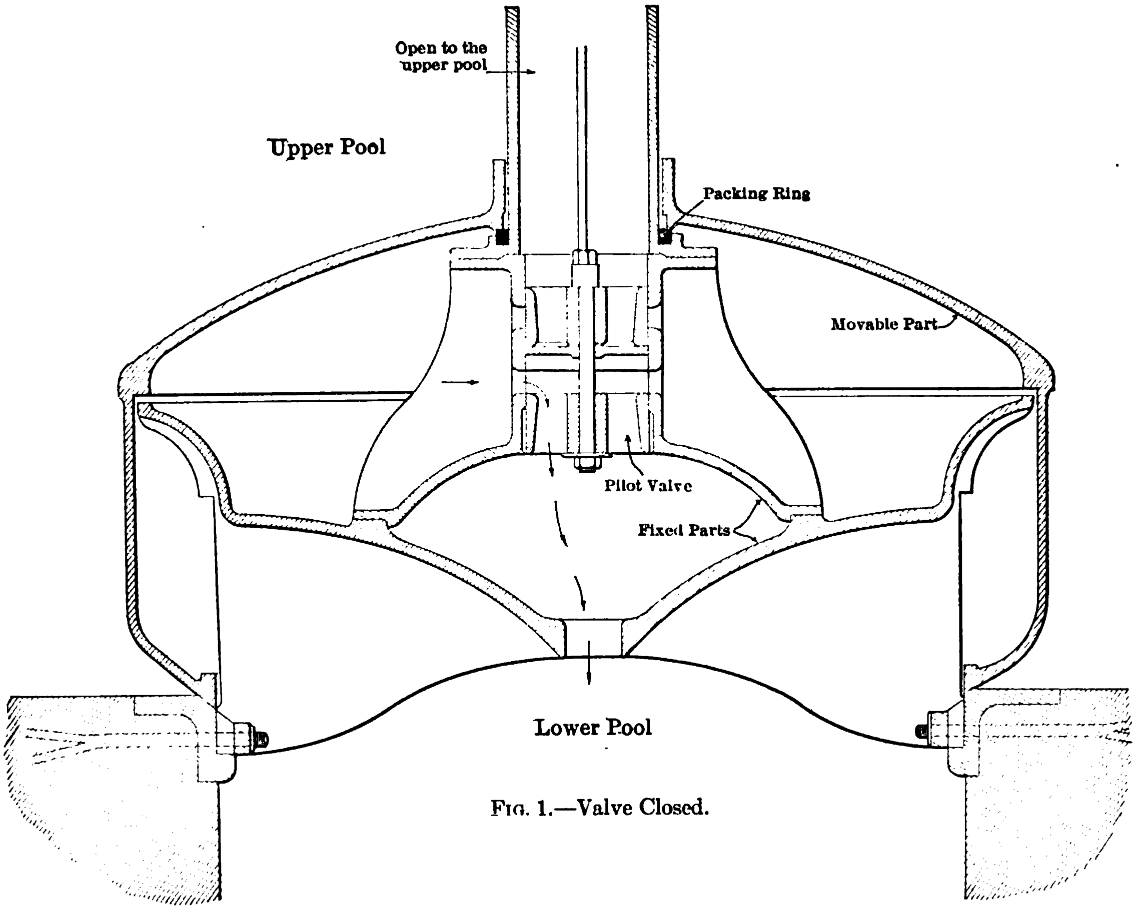


FIG. 1.—Valve Closed.

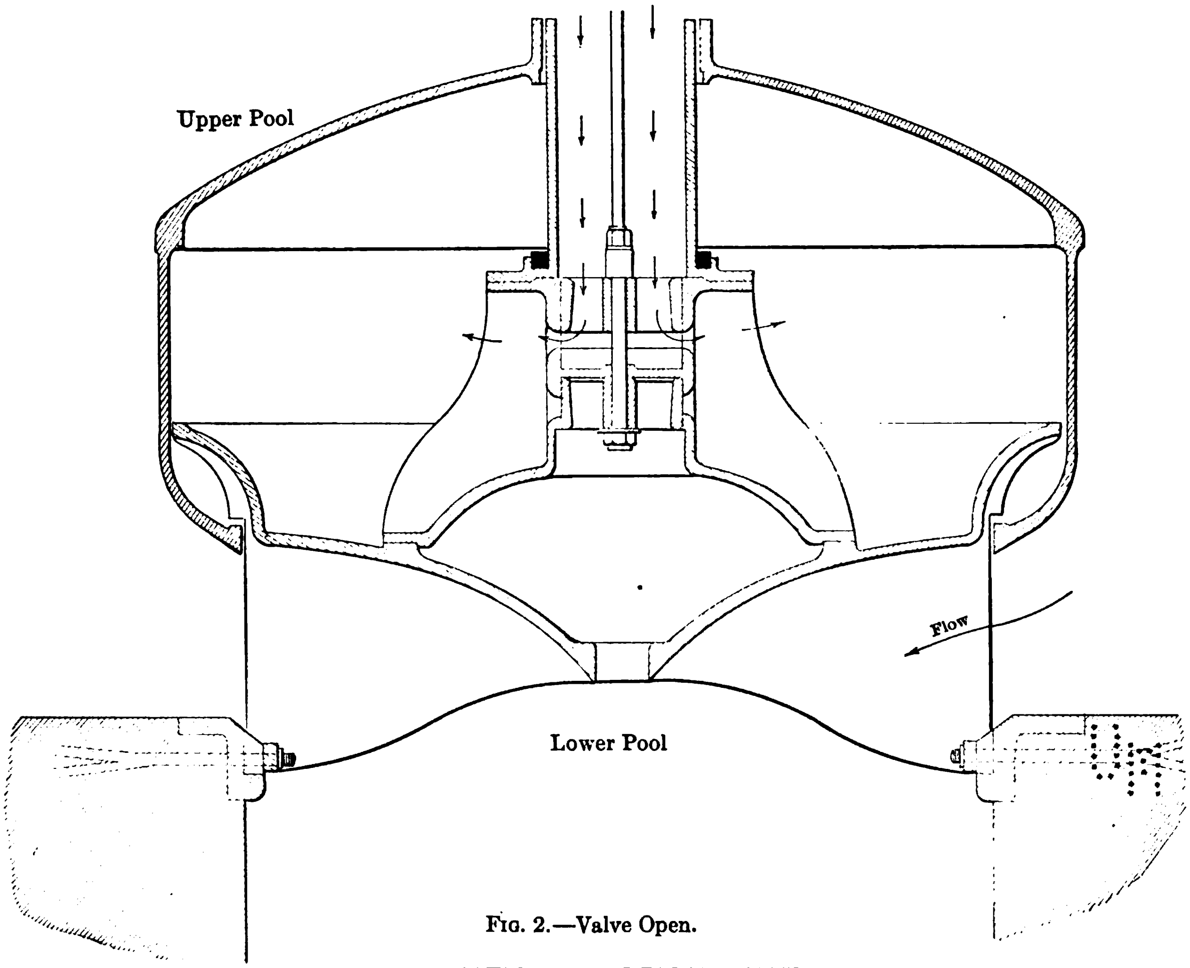


FIG. 2.—Valve Open.

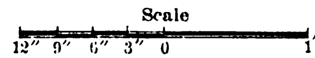
AUTOMATIC CYLINDRICAL VALVE

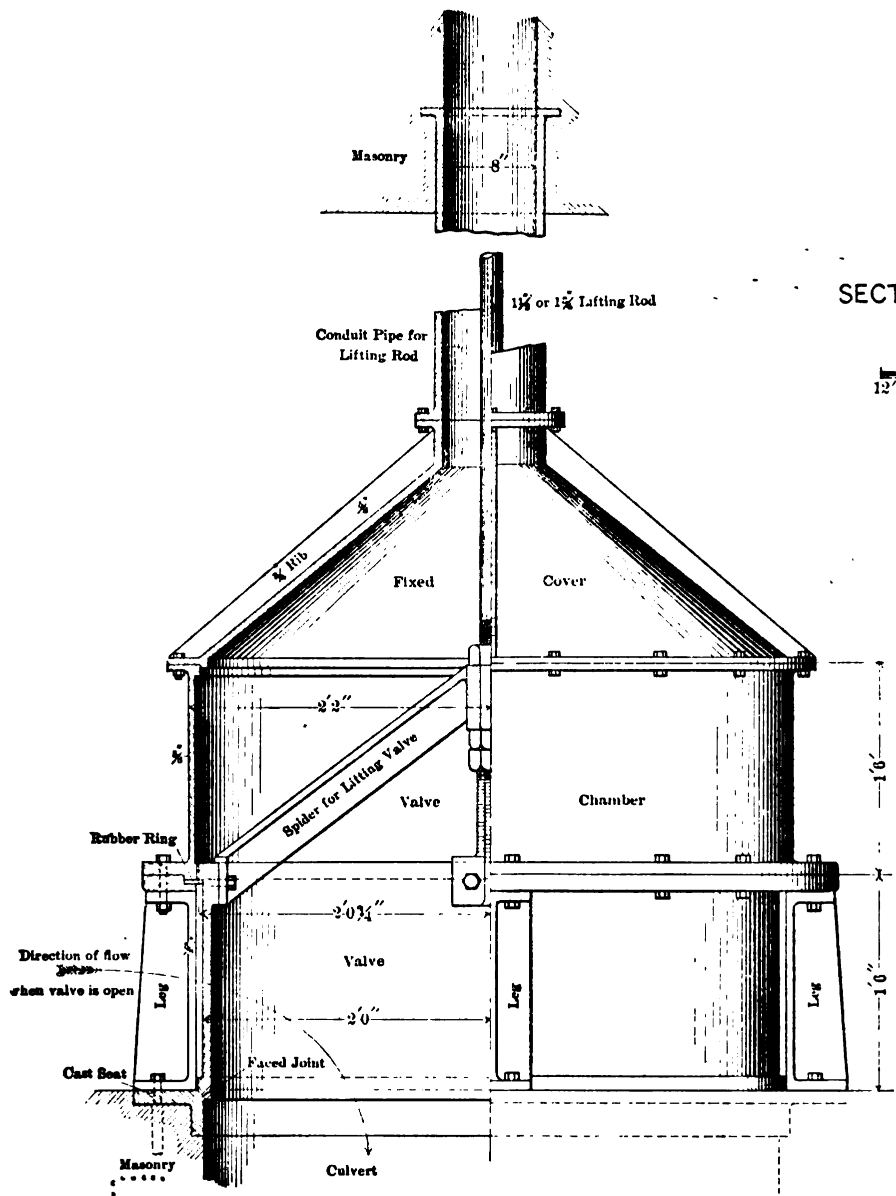
(CLUETT TYPE.)

PL. 52A.

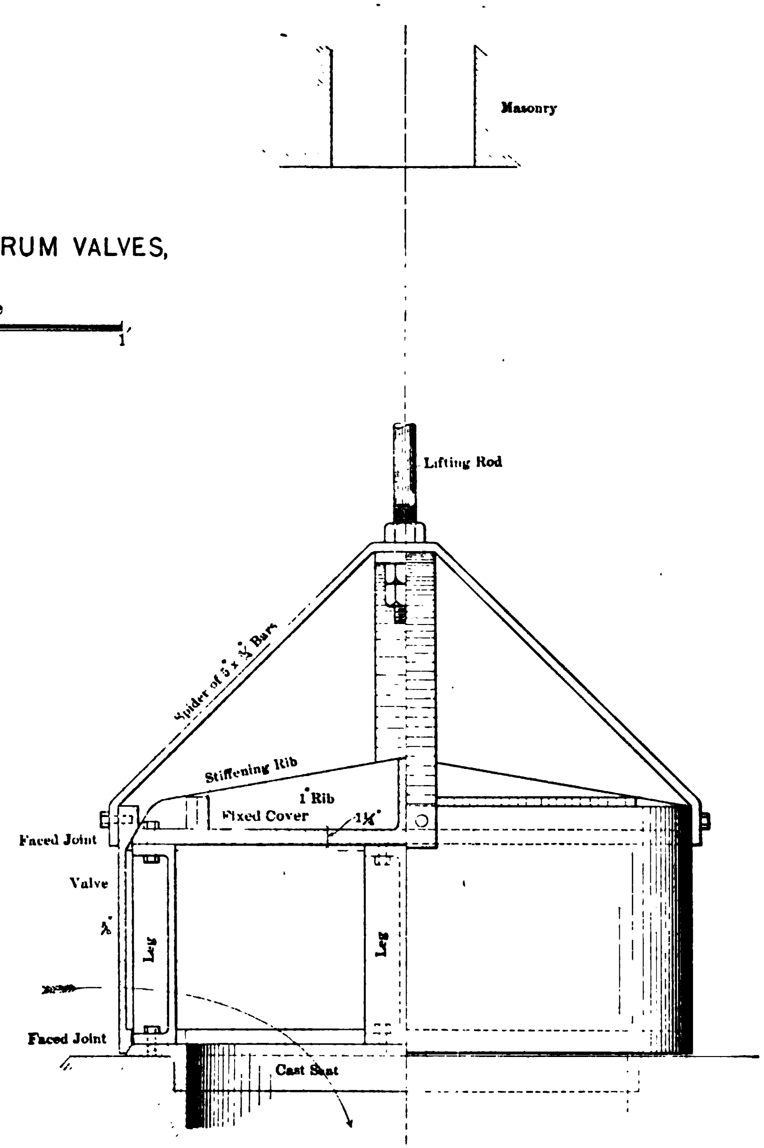
(Reference, p. 497.)

SECTIONS OF DRUM VALVES,





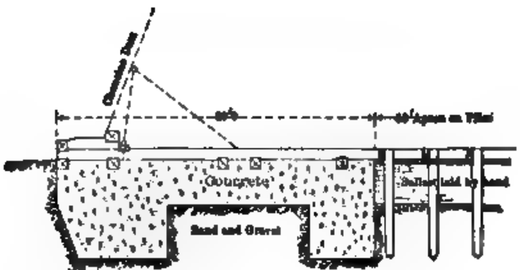
SECTIONAL ELEVATION OF DRUM OR CYLINDRICAL VALVE (FONTAINES VALVE) AS USUALLY MADE, WITH VALVE MOVING IN WATERTIGHT CHAMBER.



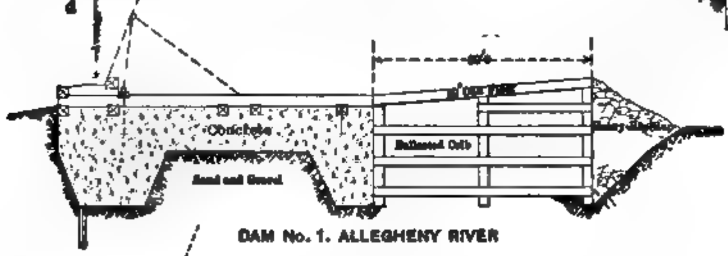
SECTIONAL ELEVATION OF PROPOSED DRUM VALVE WITHOUT WATERTIGHT CHAMBER.

PL. 53.
(Reference, p. 497.)

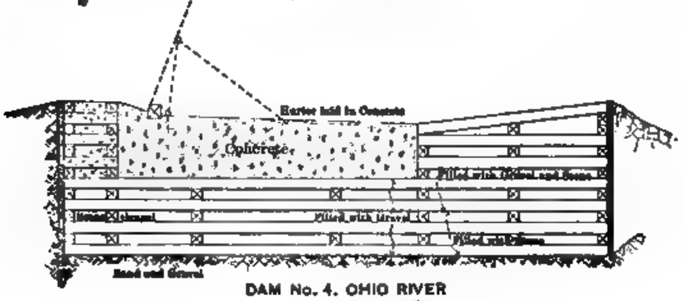
MU



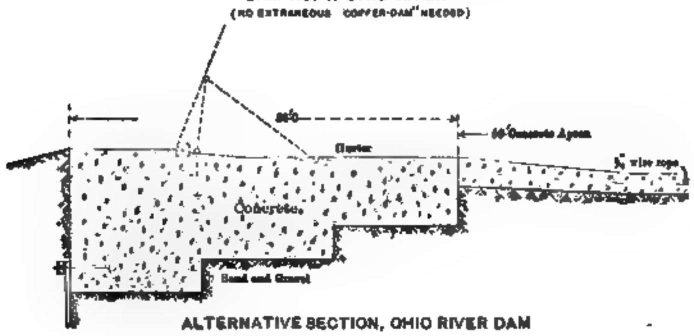
DAM No. 6. OHIO RIVER



DAM No. 1. ALLEGHENY RIVER

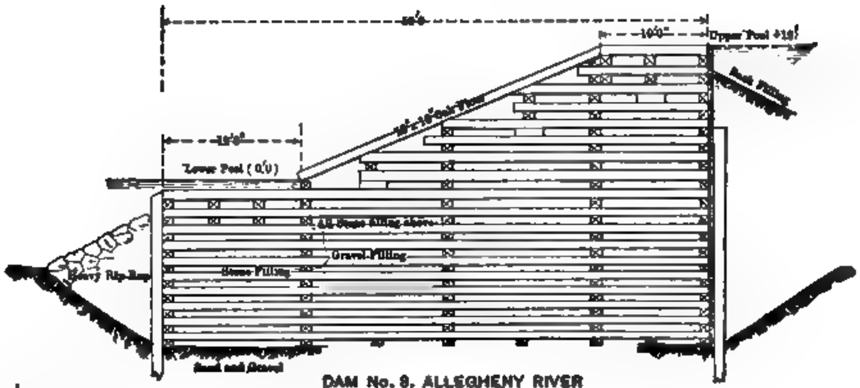


DAM No. 4. OHIO RIVER
(NO EXTRANEOUS COFFER-DAM NEEDED)



ALTERNATIVE SECTION, OHIO RIVER DAM

(See Pl. 67 for Dams Nos. 3 and 5.)

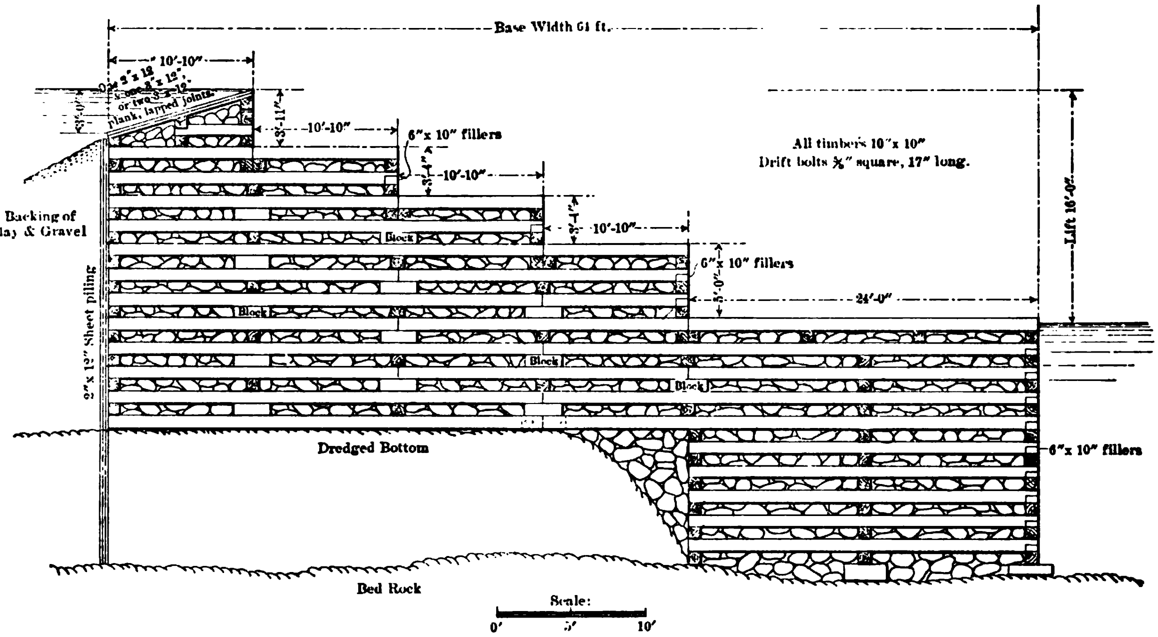


DAM No. 8. ALLEGHENY RIVER
(1906)

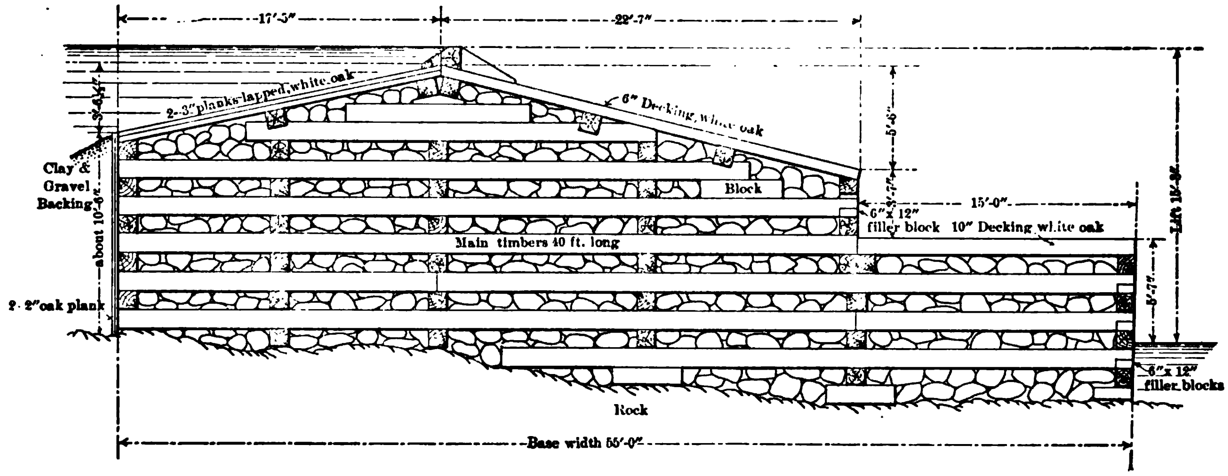
DAM No. 2. MONONGAHELA RIVER
(NO EXTRANEOUS COFFER-DAM NEEDED)
(1905)

SECTIONS OF OHIO RIVER AND OTHER DAMS.

Pl. 54.
(Reference, pp. 506, 547, 595, and elsewhere.)



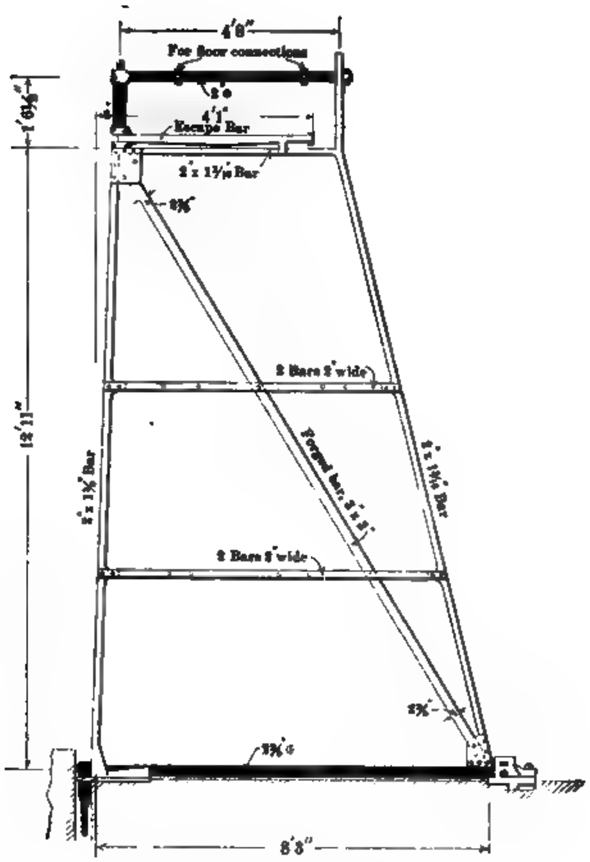
Section of Step Dam for Rivers of High Floods, after Examples Built in 1896–1900.



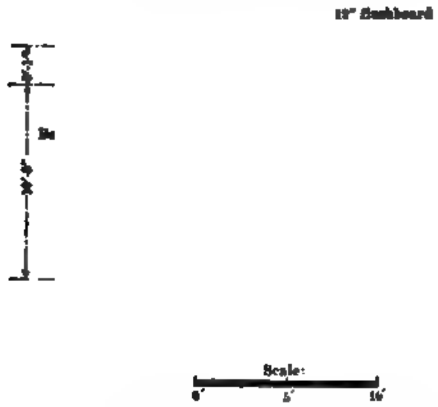
Slope Dam with Comb-stick and Long Up-stream Slope.
(Dam No. 2, Green River, Ky., 1897.)

EXAMPLES OF FIXED DAMS IN AMERICA, OF TIMBER CRIBS FILLED WITH STONE.

Pl. 55.
(Reference, p. 508 and after.)



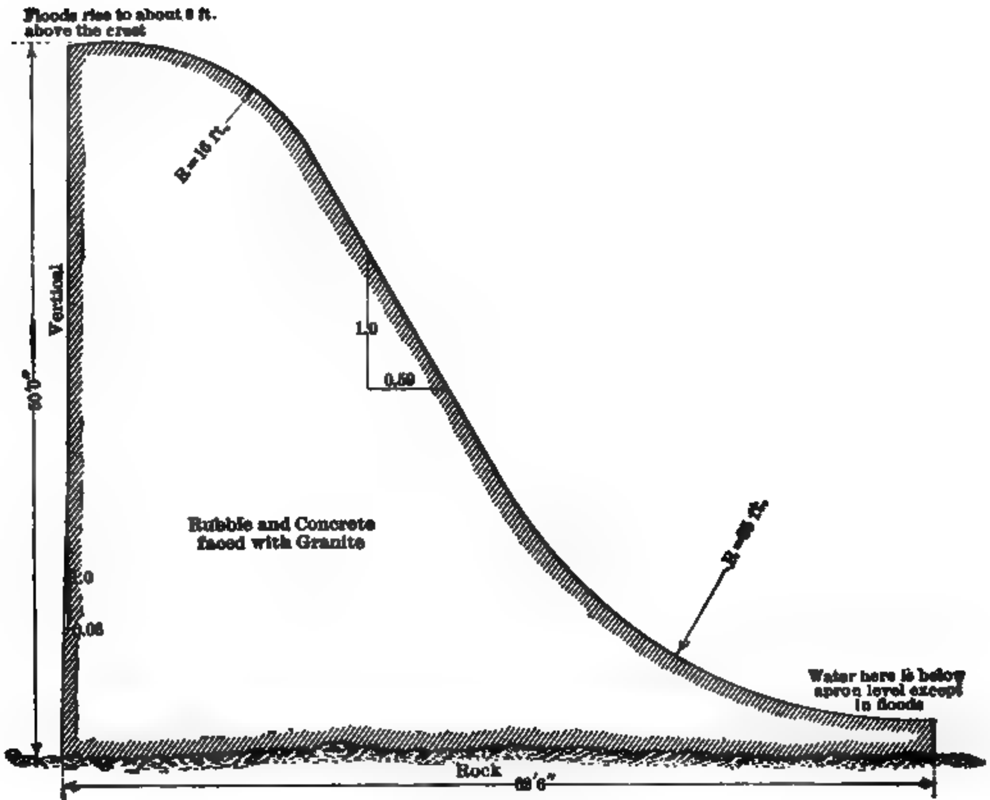
PASS TRESTLE OF THE KLECAN NEEDLE DAM ON THE RIVER MOLDAU, BOHEMIA, 1899.
(Trestles are spaced 1½ meters apart.)



SECTION OF TIMBER DAM AS USED ON THE FOX RIVER, WISCONSIN, 1898.
(Ordinary flood range, about 3 feet.)

PL. 56.

(Reference, pp. 512 and 581.)



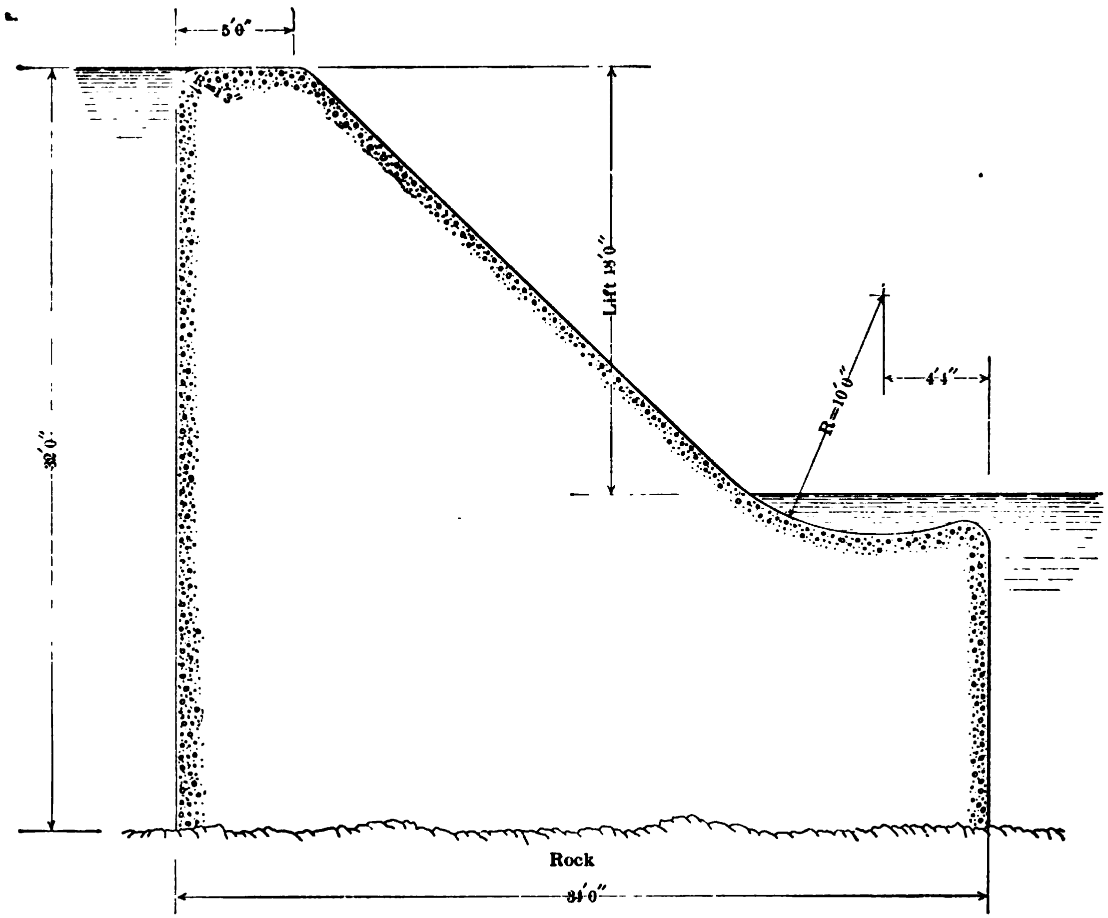
SPIERS FALLS DAM
HUDSON RIVER (1902)

CROSS SECTION OF DAM NO. 11
MONONGAHELA RIVER (1903)

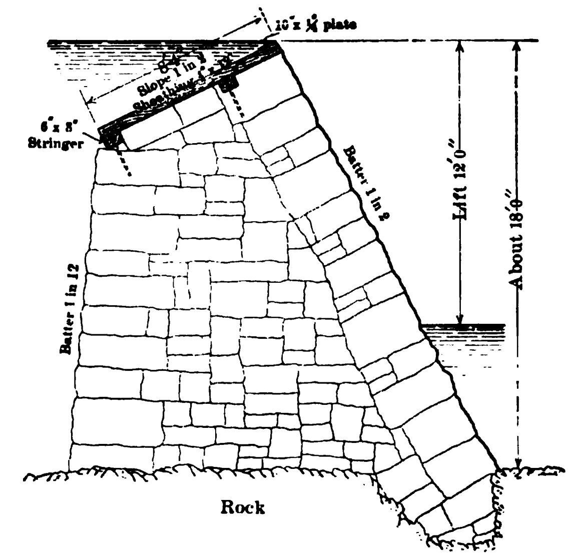
EXAMPLES OF MASONRY DAMS IN AMERICA.

Pl. 57.

(Reference, p. 515 and after.)



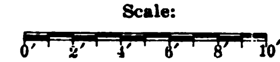
CONCRETE DAM.
No. 9, Kentucky River, Ky. 1902
Length 250 ft. Proportions about 1 of Portland Cement to 12 of sand and limestone. Facing 1 to 2, 1⅜ inches thick.



RUBBLE MASONRY DAM.
(No. 4, Black Warrior River, Alabama. 1902.)
Length 640 ft. laid in Portland Cement mortar.

EXAMPLES OF MASONRY DAMS IN AMERICA.

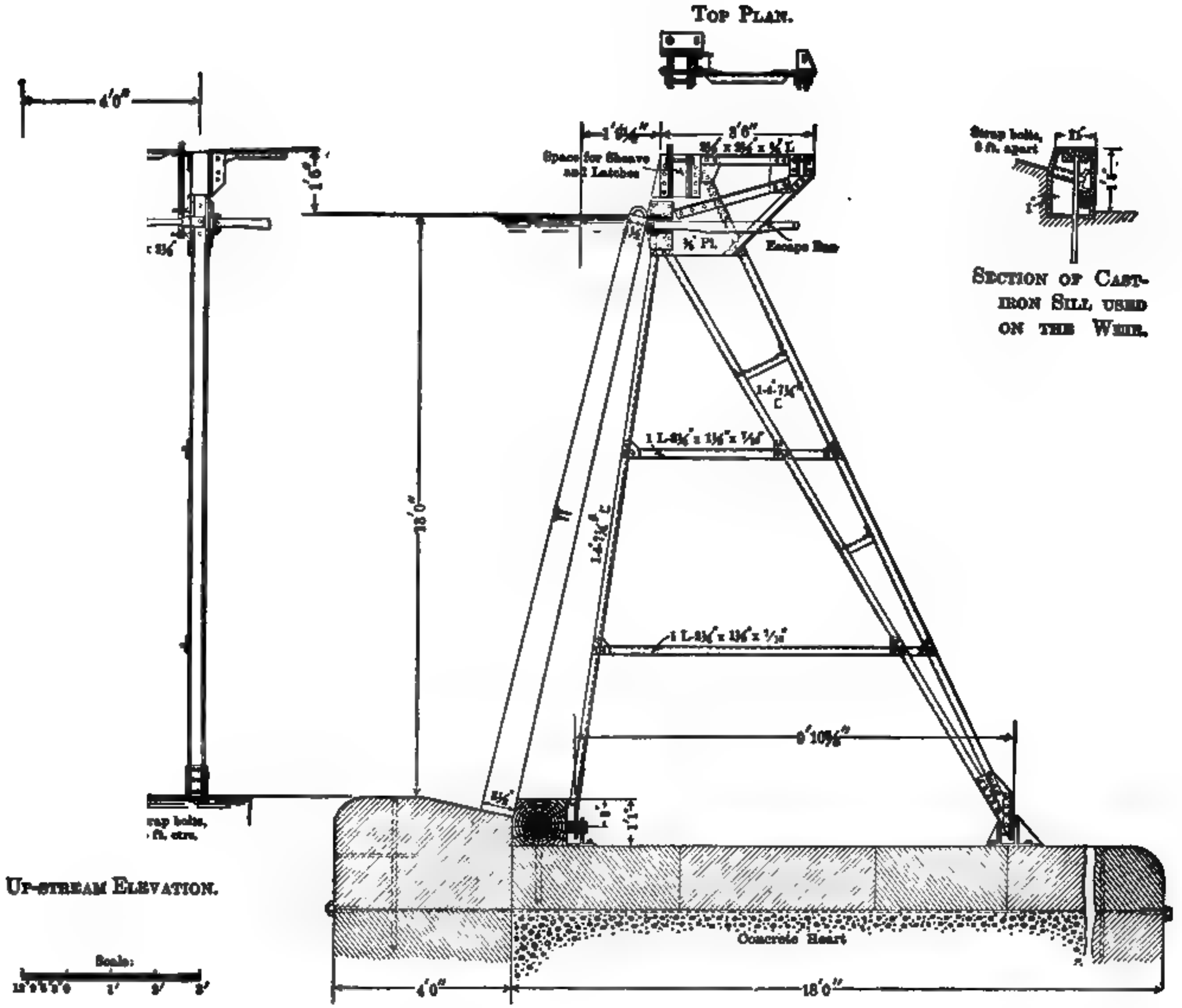
Scale:



Pl. 57A.
(Reference, p. 515 and after.)

PLATE 57b

PLATE 57b



Pass Trestle and Needle, Louisa Dam, Big Sandy River, Ky., 1896.

Pl. 58.
(Reference, p. 563 and after.)

PLATE 59

Tripping [illegible]

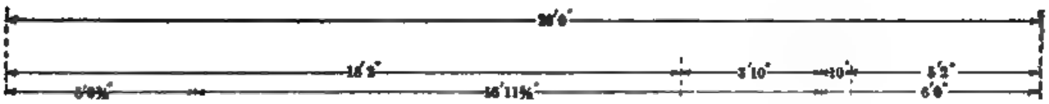
DOWN-STREAM ELEVATION
OF BUTTRESS

DAM NO. 1, BIG SANDY

DETAILS OF FOUNDATION

PLAN OF SILL AND TRESTLE ANCHORAGES

SECTION THROUGH CENTER LINE OF BUTTRESS.

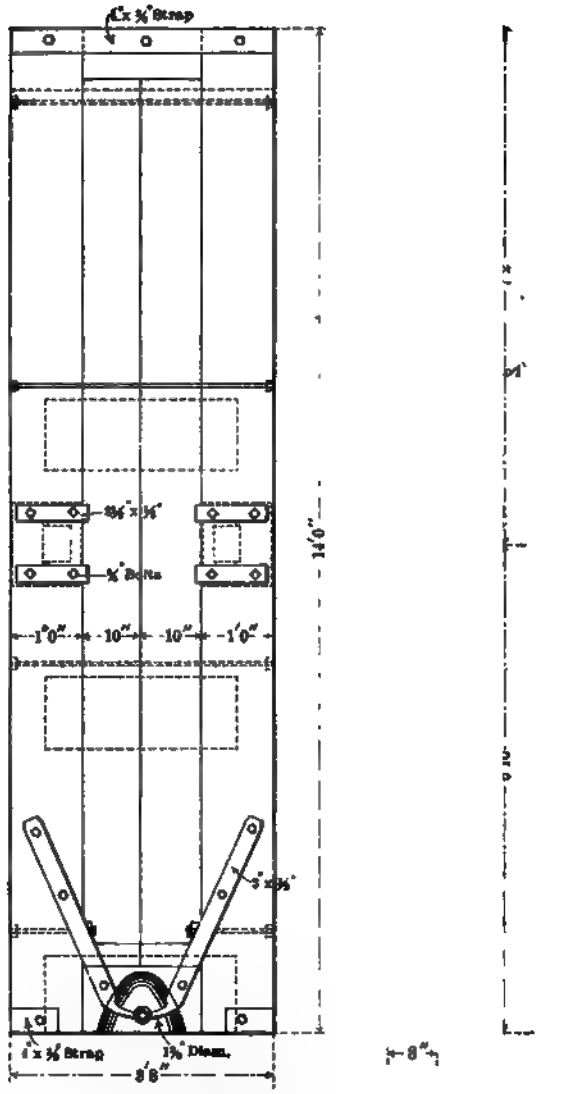


RIVER, W. VA. AND KY.

S AND SUPERSTRUCTURE.

PL. 59.

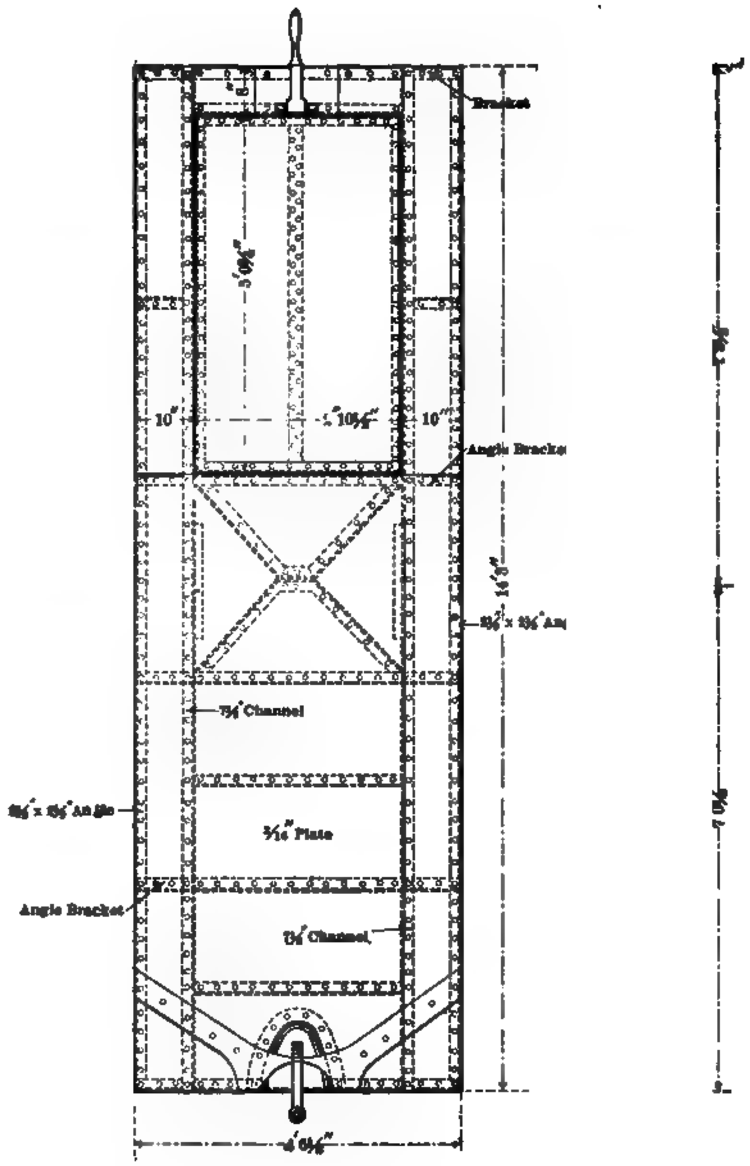
(Reference, pp. 569 and 590 and after.)



ELEVATION OF UPSTREAM FACE. SECTION NEAR CENTER.

WOODEN WICKET—PASSES OF KANAWHA AND OHIO RIVERS, U.S.A.

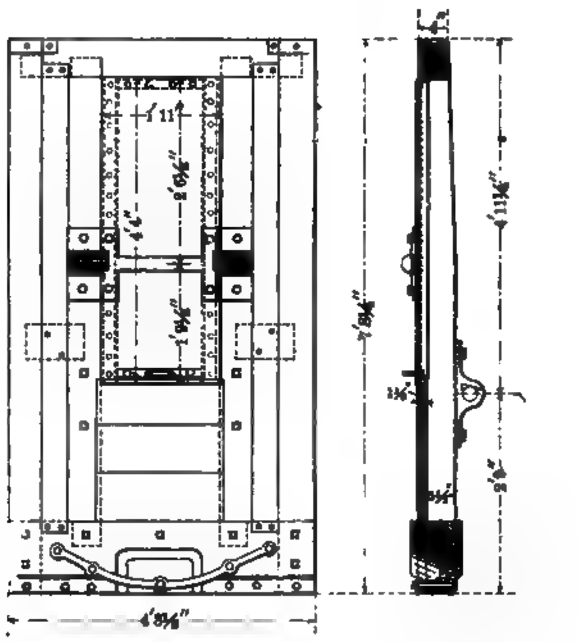
(1880-1900.)



ELEVATION OF UPSTREAM FACE. SECTION NEAR CENTER.

IRON WICKET—PASS OF LA MULATIÈRE DAM, ON THE SAÔNE, NEAR LYONS, FRANCE.

(1879.)



ELEVATION OF UPSTREAM FACE. SECTION NEAR CENTER.

WEIR WICKET USED ON THE BELGIAN MEUSE.

(About 1876.)

DETAILS OF CHANOINE WICKETS.

Scale of Feet

12" 6" 0" 1' 2' 3'

Pl. 60.

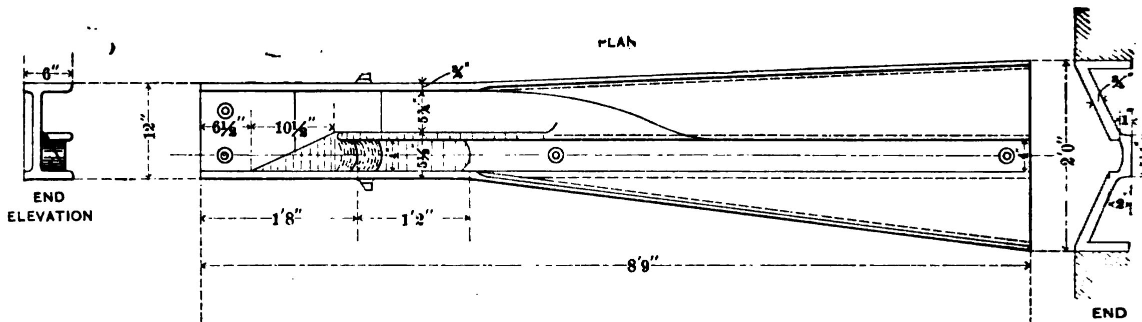
(Reference, pp. 585 and 595, and after.)

(Reference, p. 585.)

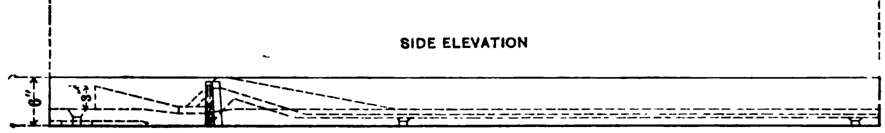
SECTION OF PASS, PORT-À-L'ANGLAIS DAM

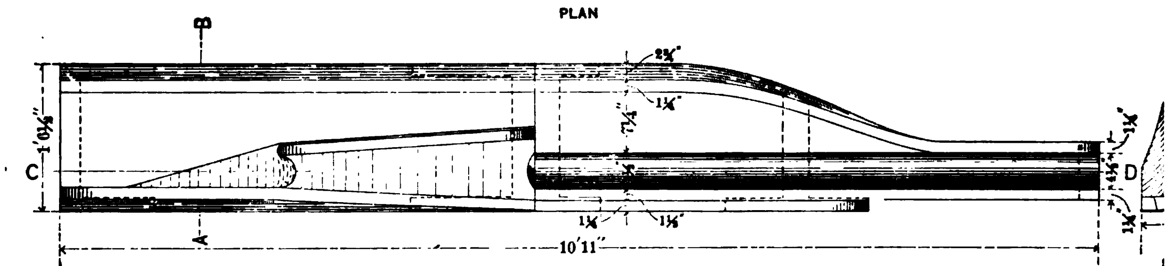
(ON THE RIVER SEINE, JUST ABOVE PARIS. THE LOWERING IS DONE BY TRIPPING BAR SHOWN ABOVE.)

PLATE 62

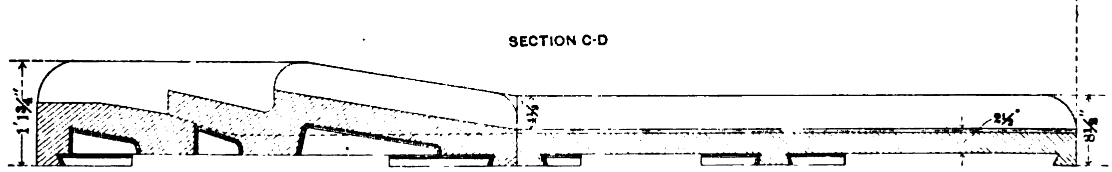


PASS HURTER, DAM No. 6, OHIO RIVER.
(1899.)

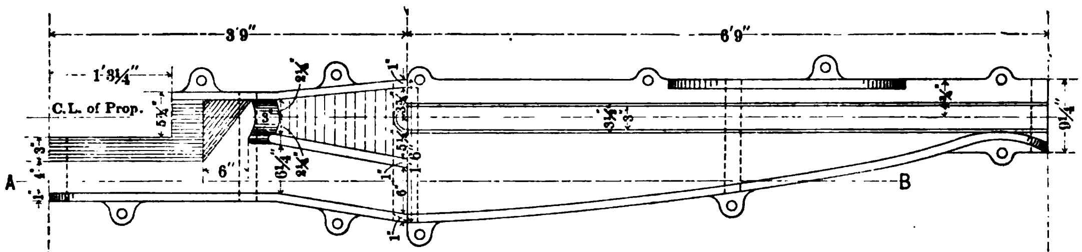




PASS HURTER, LA MULATIÉRE DAM.
(On the Saône, near Lyons, 1879.)

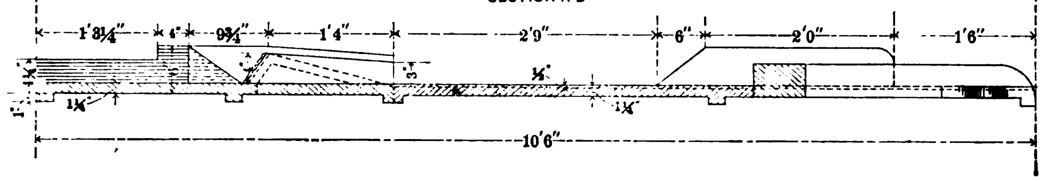


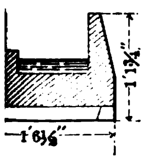
TYPES OF HURTERS USED ON



PASS HURTER, KANAWHA RIVER, W. VA.
(1896.)

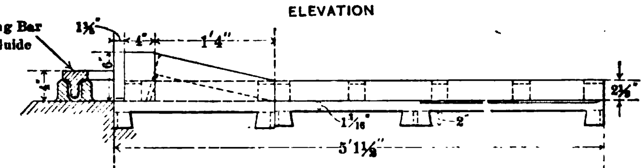
SECTION A-B



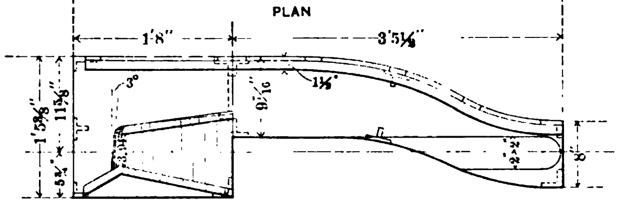


ECTION A-B

ELEVATION



PLAN



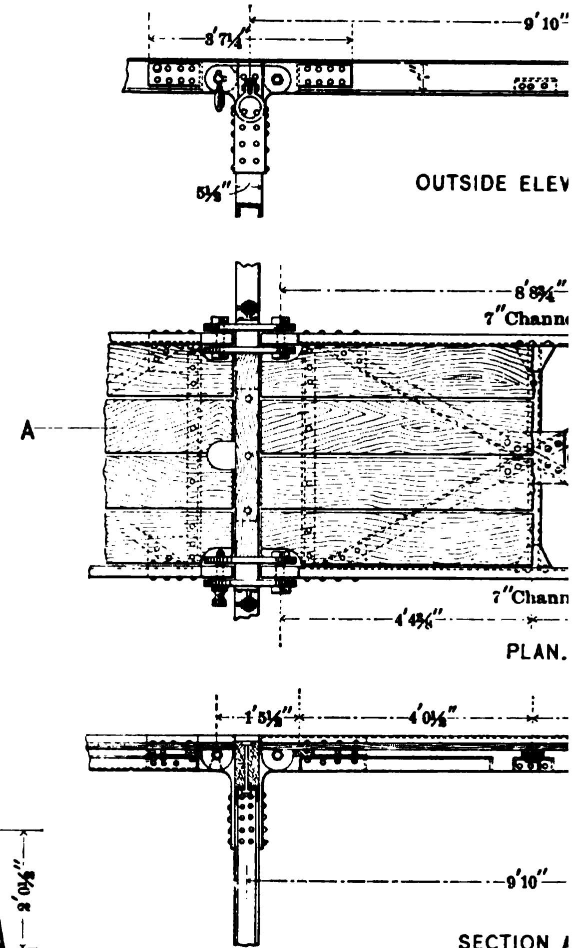
WEIR HURTER OF THE BELGIAN MEUSE.
FOR USE WITH TRIPPING BAR.
(About 1876.)

CHANOINE WICKET DAMS.

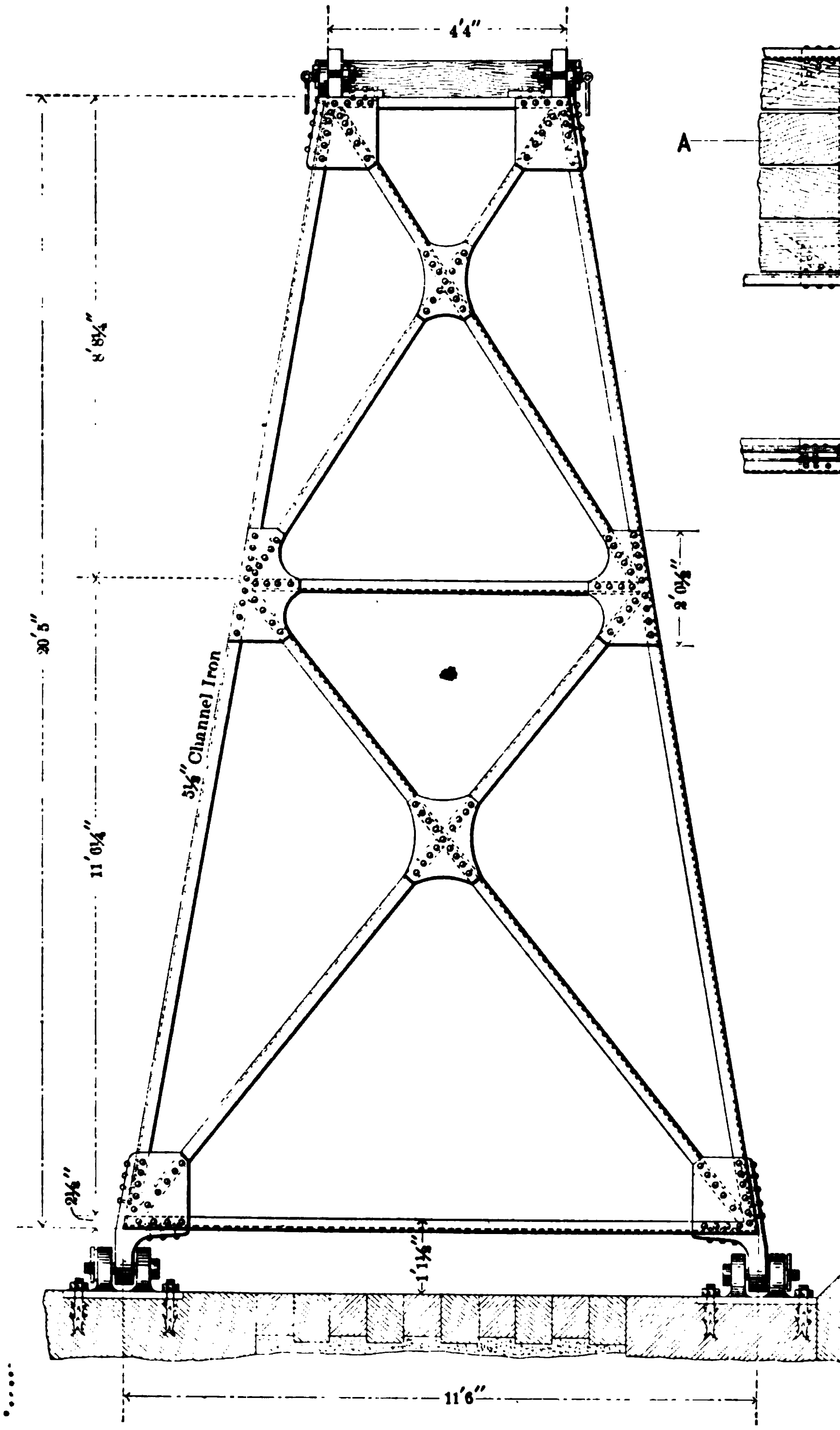
PL. 62.

Reference, p. 589.)

PLATE 63



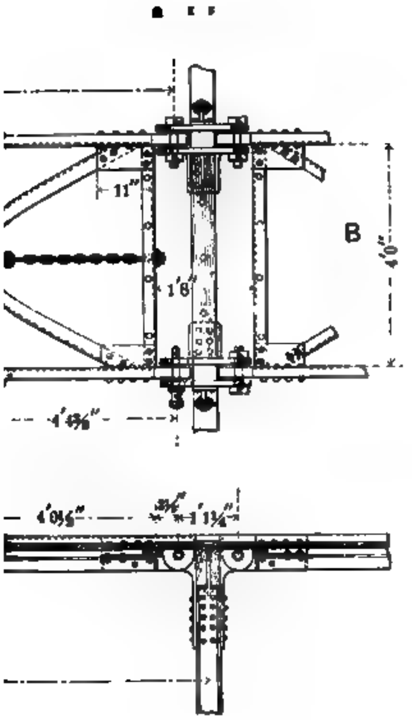
FLOORS OF LA MULA



PASS TRESTLE, LA MULATIERE DAM.
(Saône, 1879.)

TRESTLES FO
CHANOIN

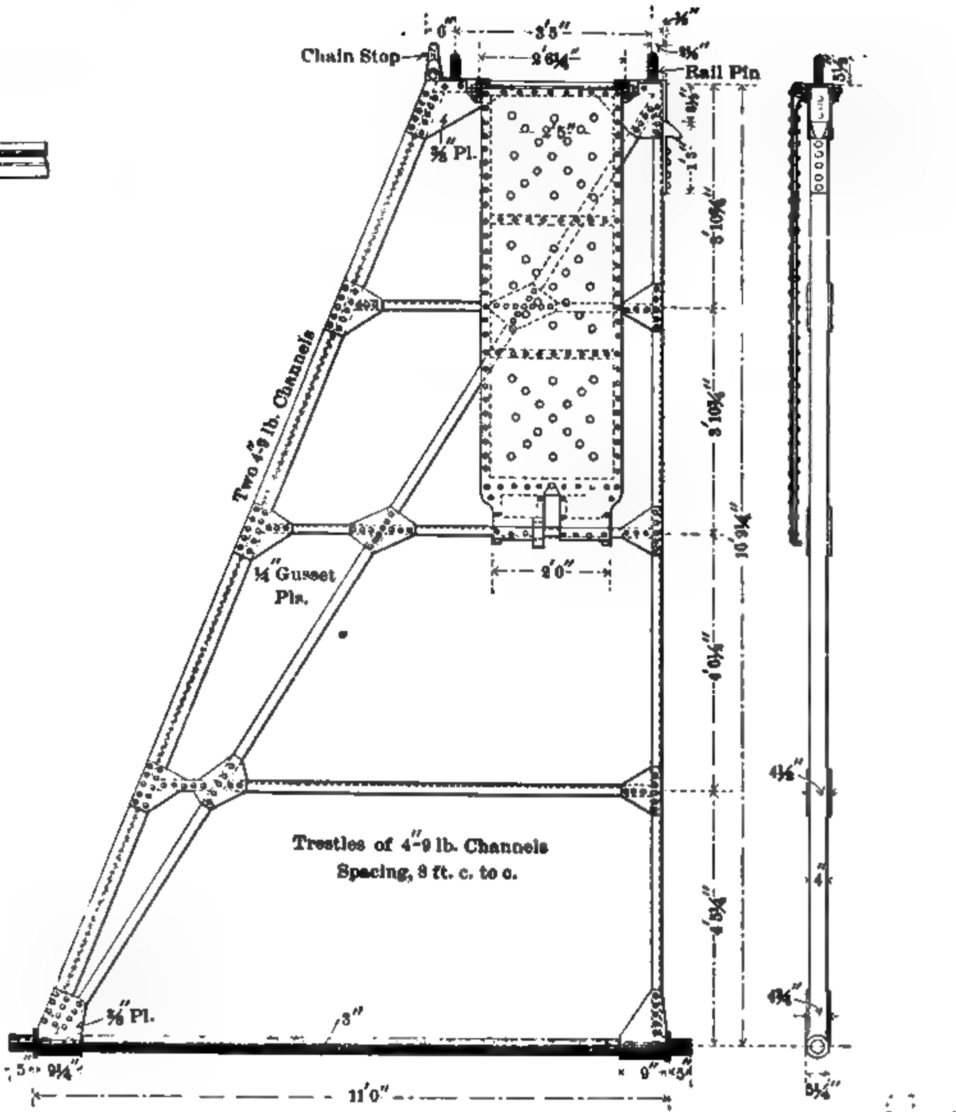
12" 6" 0"



ERE TRESTLES.

SERVICE BRIDGES,
WICKET DAMS.

Scale:

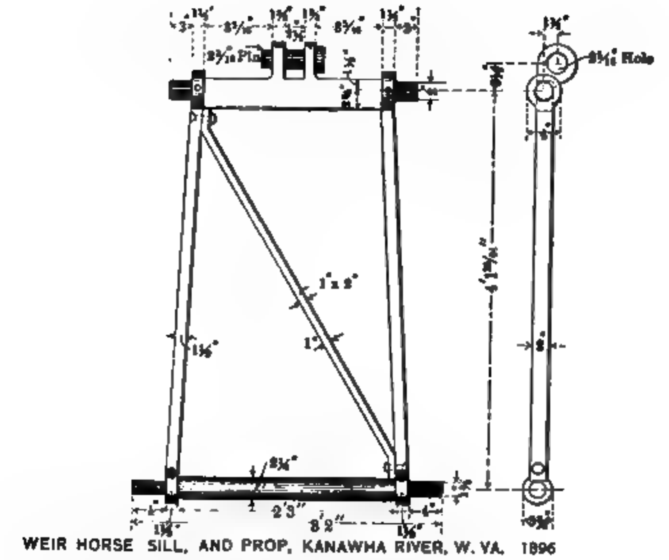


PASS TRESTLE, KANAWHA RIVER, W. VA.
(1896.)

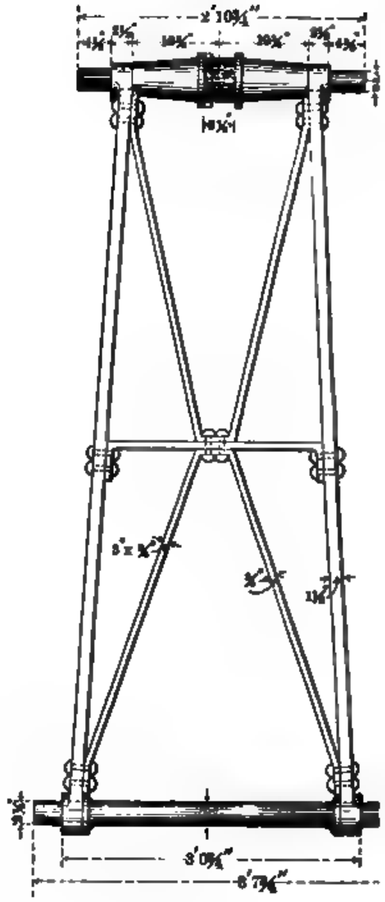
Pl. 63.
(Reference, p. 591 and after.)

PLATE 63a

PLATE 64



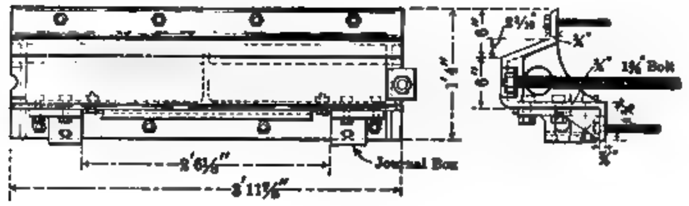
WEIR HORSE SILL, AND PROP, KANAWHA RIVER, W. VA. 1896

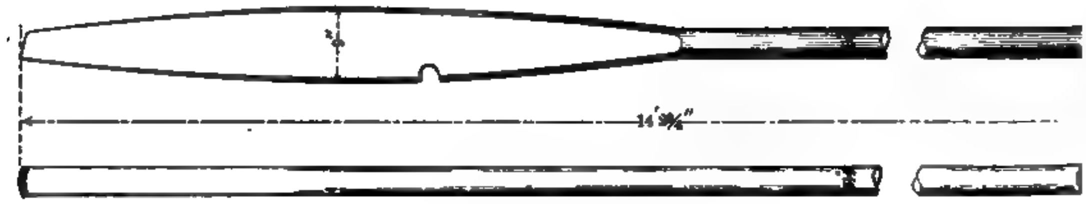


PASS HORSE, DAM No. 6, OHIO RIVER, 1899

PLAN OF SILL.

END ELEVATION.





PASS PROP, DAM No. 6, OHIO RIVER

DETAILS OF HORSES, PF

OF CHANOINE WIC

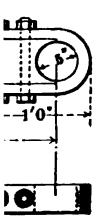
Scale:

12" 0' 1

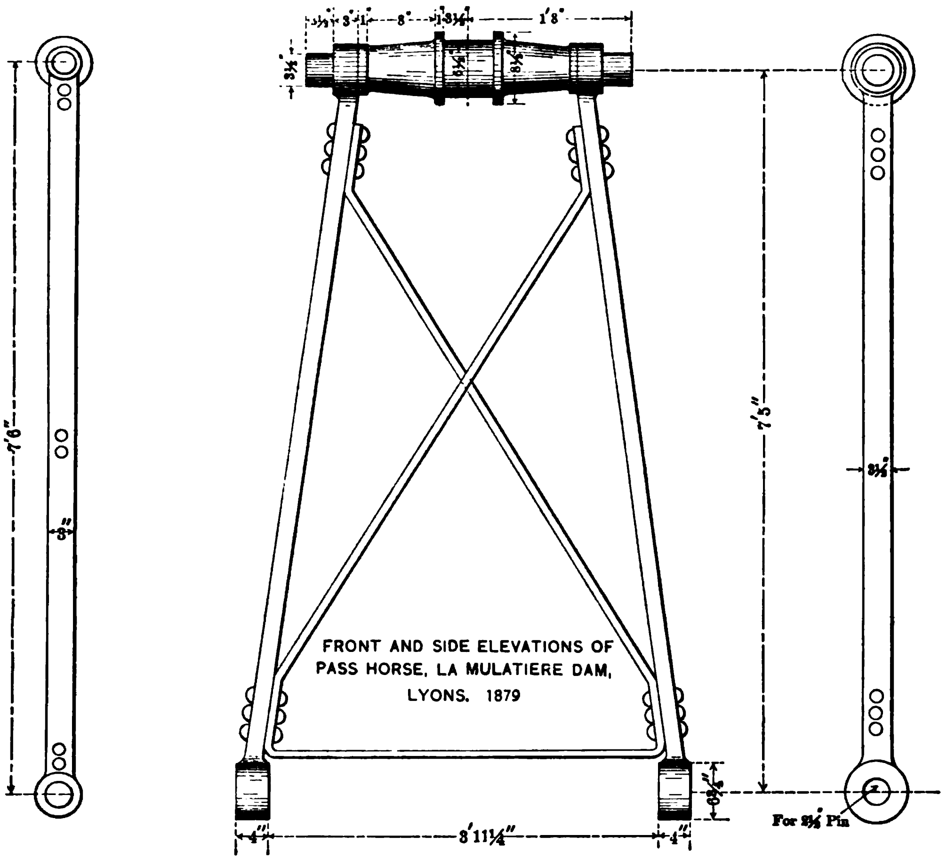
SECTION OF SILL OF LA MULATIERE DAM.

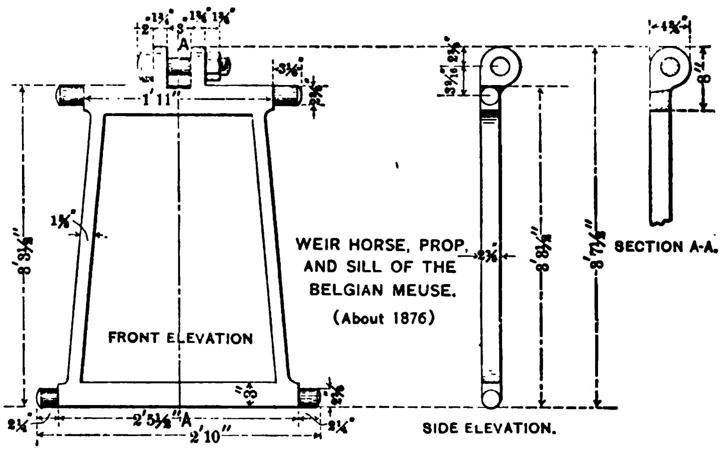
Fimber

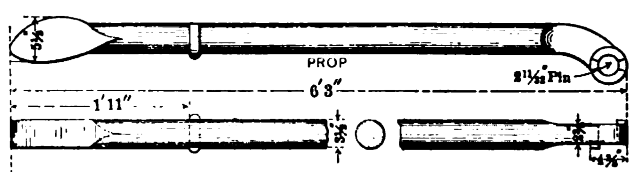
OF PASS SILL,
)AM No. 6

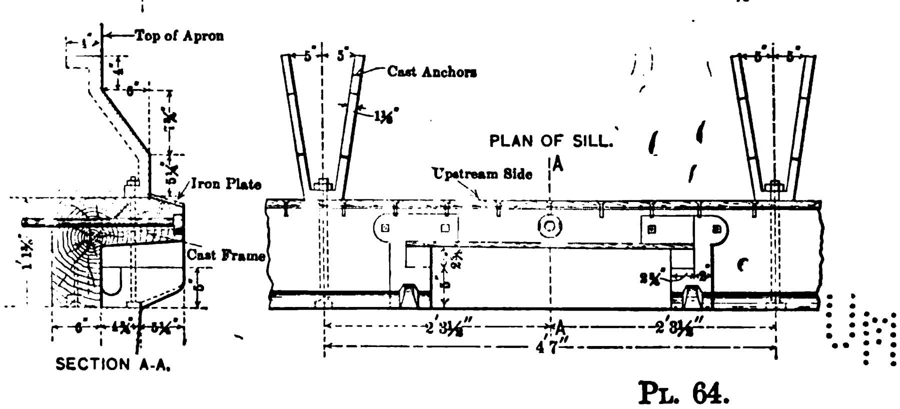


)PS, AND SILLS
ET DAMS.









PL. 64.

(Reference, p. 595 and after.)

Plan of Navigation Pass.

General Section of Navigation Pass—Center Pier in Elevation.

Scale. 0 1 2 3 4 5 10 15 20 FEET.

SECTIONS, ETC., OF PASS AND WEIR, DAM NO. 7, KANAWHA RIVER, W. VA.

PL. 65.

(Reference, p. 595.)

GENERAL PLAN AND SECTION OF THE DAVIS ISLAND DAM, OHIO RIVER, NEAR PITTSBURG, PA.

PL. 66.

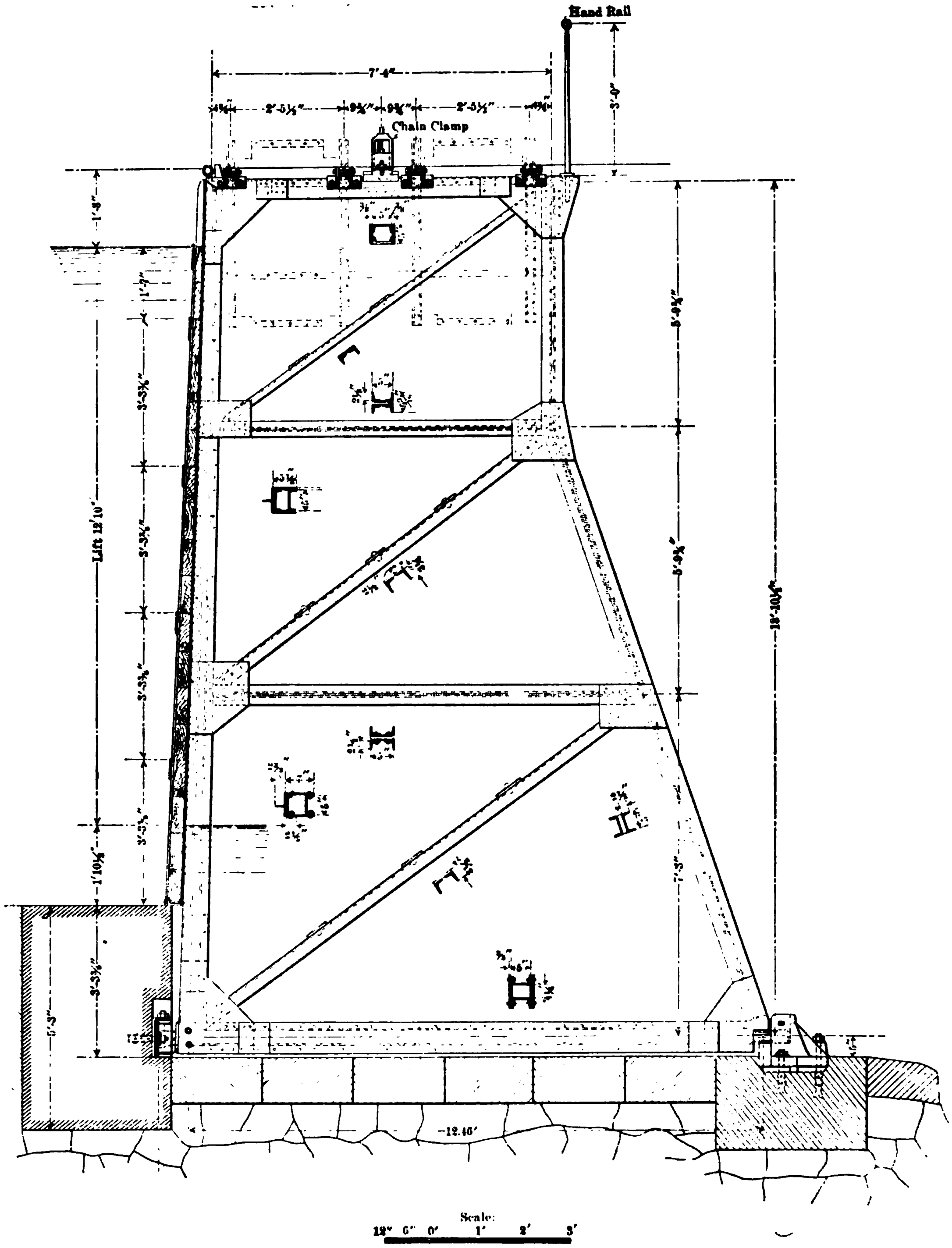
(Reference, p. 5[illegible]5.)

FOUNDATIONS OF OHIO RIVER DAMS

(See also Pl. 54.)

PL. 67.

(Reference, pp. 549, 595.)



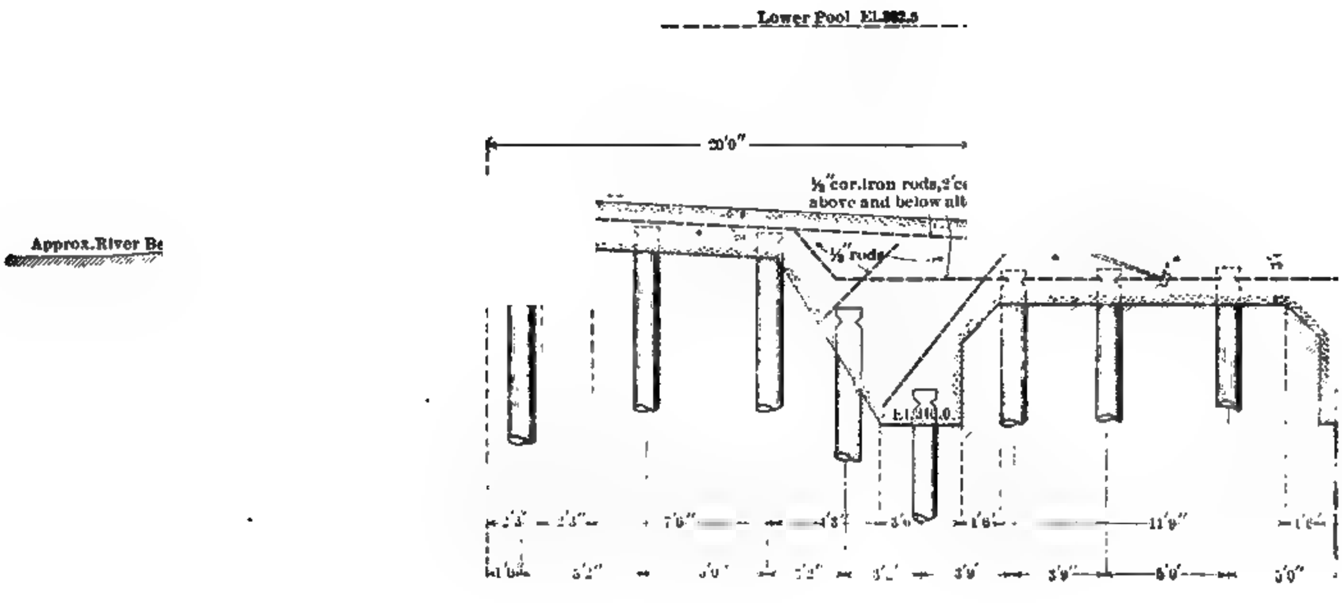
GENERAL SECTION OF THE BOULÉ DAM AT LIBSCHITZ, BOHEMIA, 1900.

Pl. 68.

(Reference, pp. 608 and 619.)

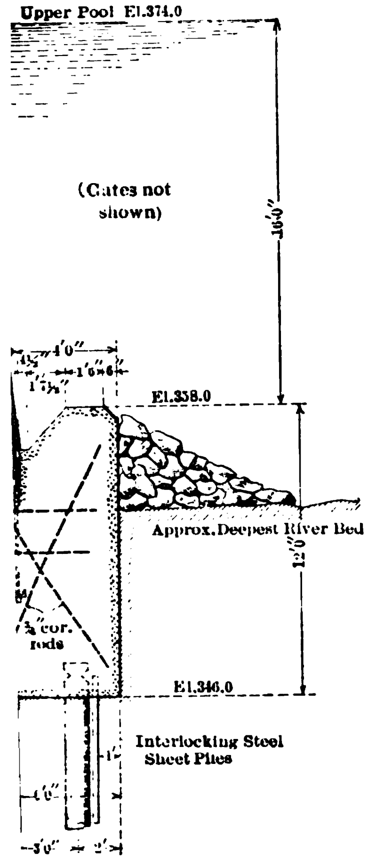
PLATE 69

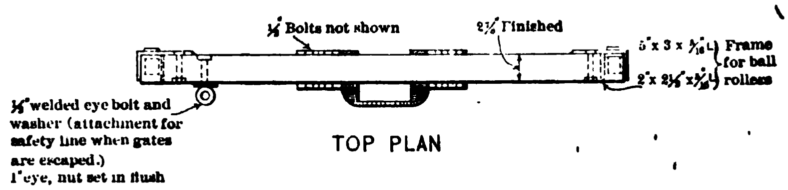
Pl. 69.
(Reference, p. 610.)



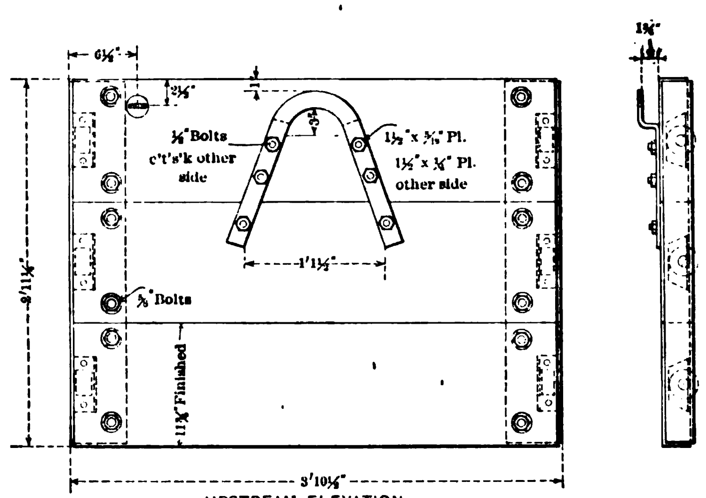
GENERAL SECTION OF FOUNDATION.

DETAILS OF DAM NO.

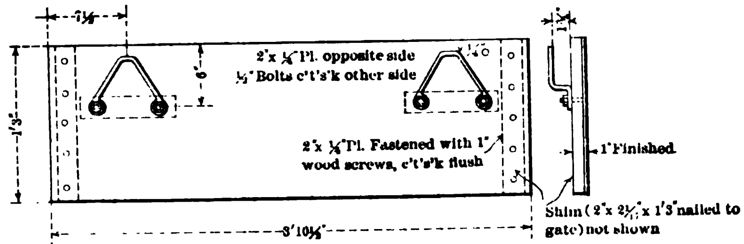




TOP PLAN

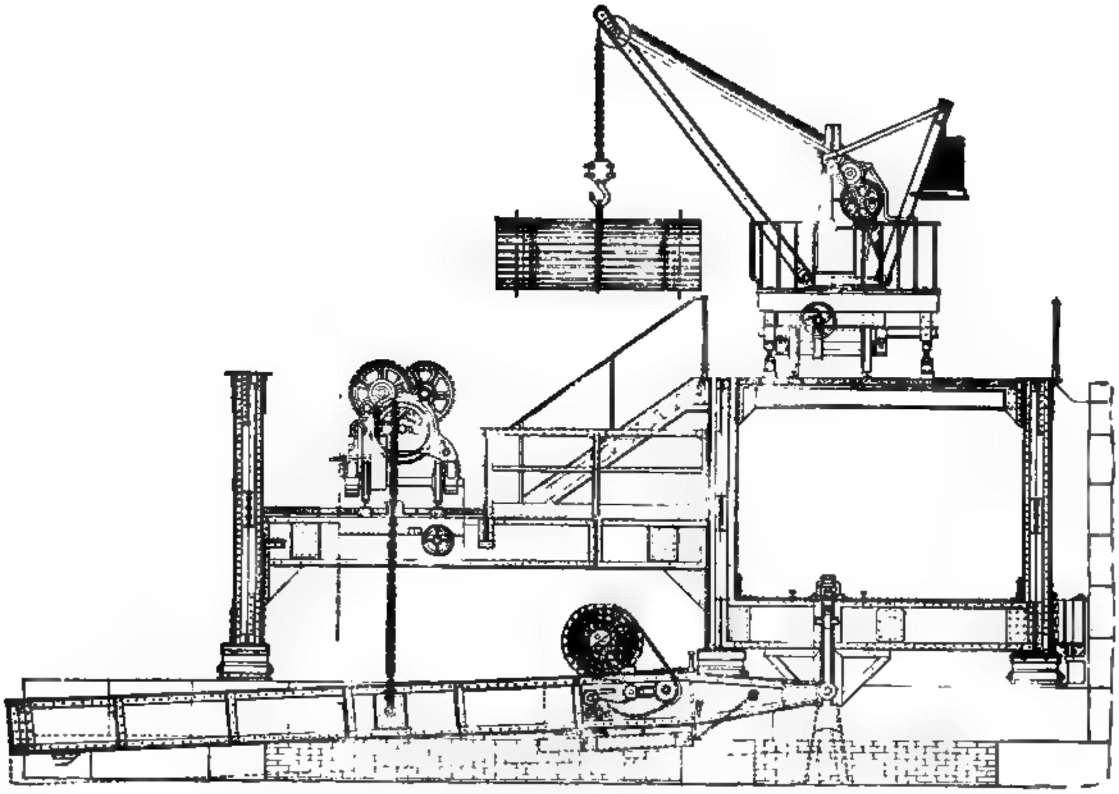


UPSTREAM ELEVATION
ROLLING GATE FOR 4 LOWER TIERS



UPSTREAM ELEVATION
SLIDING GATE FOR TOP TIER

TRINITY RIVER, TEXAS.



Section through Bridges, Showing Operating Winches.
(The curtains are not removed, except for repairs.)

DETAILS OF THE
POSES DAM
(LOWER SEINE)

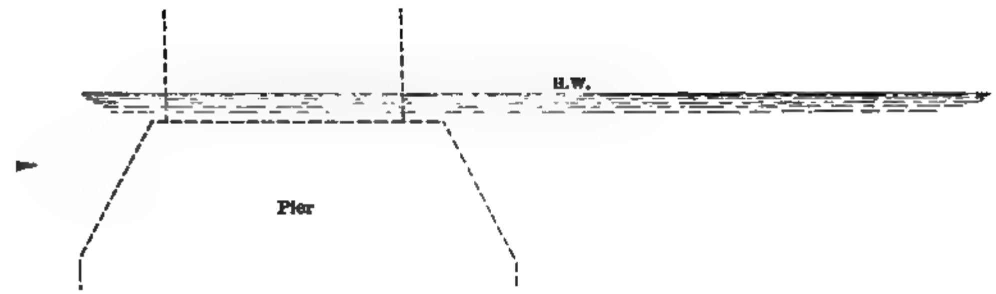
Elevation of Curtains and Uprights, Looking Down-stream.
(The curtains extend across two bays.)

Pl. 70.
(Reference, pp. 626–628.)

DETAILS OF UPRIGHTS AND CURTAINS, POSES DAM (LOWER SEINE).

Pl. 71.

(Reference, pp. 620–628.)



CROSS SECTION
WITH DAM IN POSITION

CROSS SECTION
WITH DAM RAISED

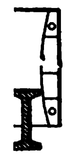
TYPICAL DESIGN OF BRIDGE DAM WITH GATES, AS USED ON THE MOHAWK RIVER, N. Y.

(NEW YORK STATE BARGE CANAL.)

PL. 72.

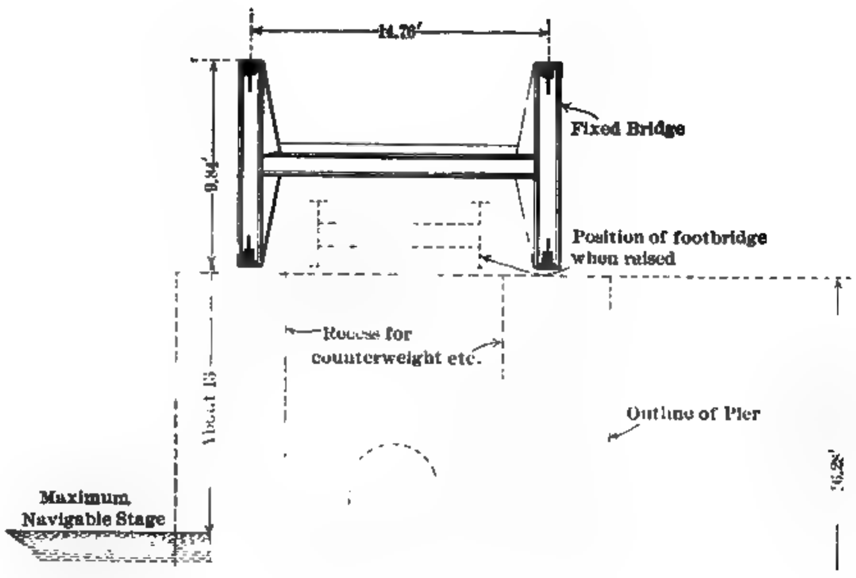
(Reference, p. 632 and after.)

PLATE 73



PLAN OF SHOE

xd 632.)

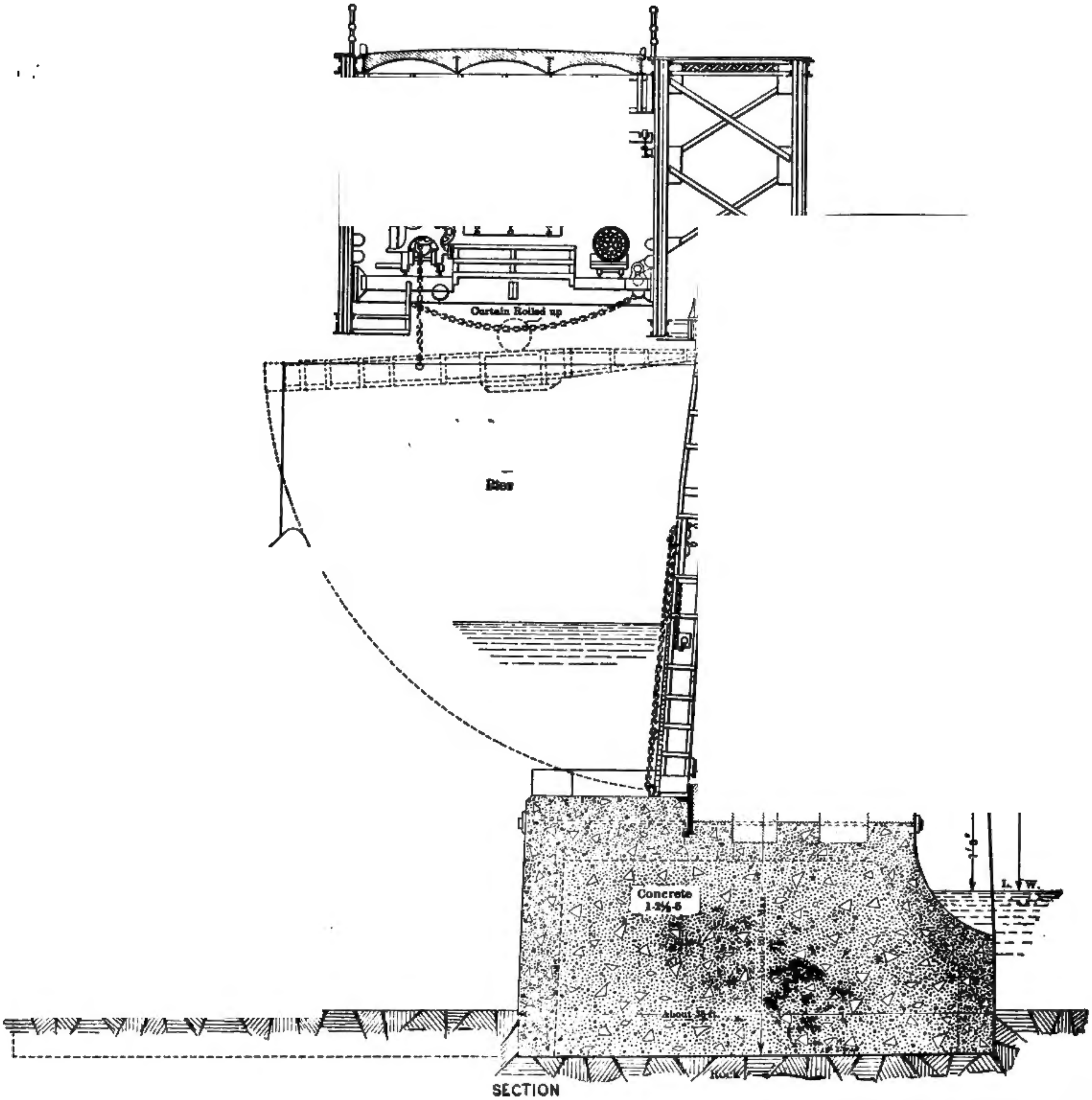


(No round or sheet piles were used)
SECTION THROUGH PASS AT CREIL DAM
(RIVER OISE, 1901)

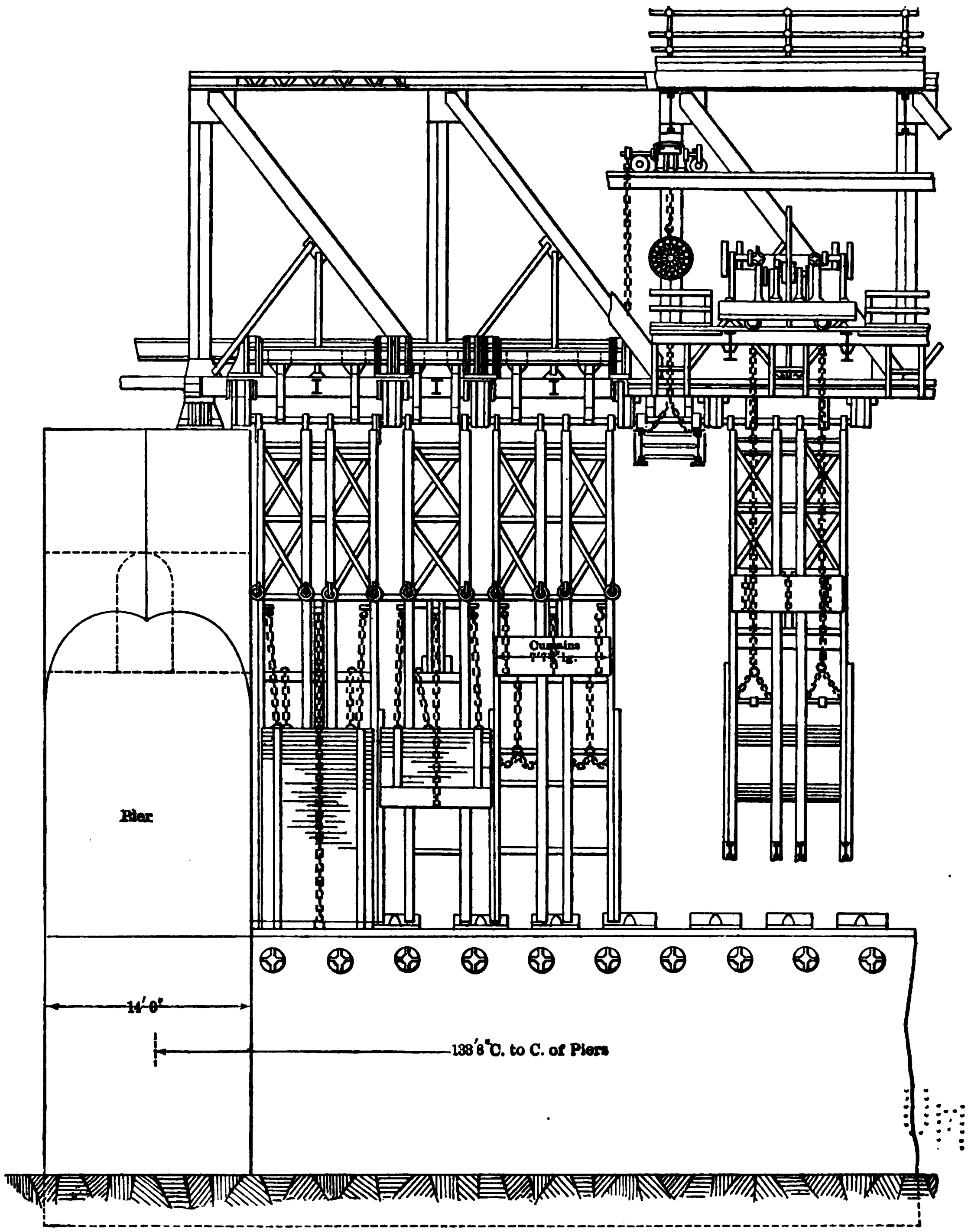
PLAN OF SHOE

PL. 74.
(Reference pp. 625 and 632.)

PLATE 75



BRIDGE DAM AT ST. ANDREW'S RAPIDS, RED R



ELEVATION LOOKING DOWNSTREAM.

ER, MANITOBA.

PL. 75.

(Reference, p. 638.)